Appendix 2- Attachment 1

Lower San Joaquin Feasibility Study
Littlejohn Creek frequency analysis and hydrographs

Lower San Joaquin River feasibility study: Littlejohn Creek frequency analysis and hydrographs

June 23, 2011

US Army Corps of Engineers Sacramento District
W91238-09-D-0004, TO 0004

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Engineer's certification

I, Michael Konieczki, hereby certify on 6/23/2011 that I am a professional engineer licensed in the state of California and that the accompanying report was prepared by me or under my supervision.

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Executive summary

Situation

In the lower San Joaquin River feasibility study (LSJR FS) the Sacramento District of the US Army Corps of Engineers (Corps) and the San Joaquin Area Flood Control Agency (SJAFCA) are studying alternative flood risk reduction measures that will provide protection against a flood with a probability of exceedence in any given year equal 0.005 (i.e., a "200-year flood").

The LSJR FS includes hydrologic analyses of the study region. This same region is also being studied in conjunction with a separate project to map the floodplains adjacent to the federal-state levee system in the Central Valley. Because the products of the various hydrologic analyses being conducted in the lower San Joaquin River basin will be used for several purposes by multiple agencies and stakeholders, the firms and agencies involved are using consistent analytical procedures and methods where possible. These procedures are specified in the *Sacramento and San Joaquin river basins: Procedures for hydrologic analysis* (hereinafter, *Procedures document*) and the *Central Valley hydrology study (CVHS): Technical procedures document* (hereinafter, *Technical procedures document*). Attachment 1 provides a table that explains how the procedures detailed in the present document align with the procedural steps detailed in the *Procedures document* and the *Technical procedures document*.

In this report we detail our hydrologic analyses at 2 sites on Littlejohn Creek: (1) Farmington Reservoir, and (2) Farmington Reservoir's operation point at Farmington, CA. These sites are shown in Figure 1.

Tasks

Our tasks were to: (1) develop a regulated flow-frequency curve and associated volumes at each location, and (2) derive an "expected" outflow hydrograph at Farmington Reservoir.

Actions

To complete the tasks above, we:

- Developed unregulated volume-frequency curves at Farmington Reservoir and Farmington, CA, following the procedures in *Guidelines for determining flood flow frequency, Bulletin 17B* (IACWD 1982) and EM 1110-2-1415 (USACE 1993) and using a regional skew provided by the Corps.
- Simulated reservoir releases and routed historical and scaled floods, including local flows, on Littlejohn Creek using an HEC-ResSim model provided by the Corps.
- Fitted, at each location, flow transforms to the event maxima dataset identified from the unregulated flow and simulated release time series.
- Developed, at each location, a regulated flow-frequency curve and associated volumes by applying the flow transforms.
- Developed "expected" outflow hydrographs for Farmington Reservoir for 8 flood frequencies: p=0.5, p=0.2, p=0.10, p=0.05, p=0.02, p=0.01,

p=0.005 and p=0.002. (Here the term expected hydrograph refers to a Farmington Reservoir outflow hydrograph with a peak flow that matches the regulated flow-frequency curve and with associated volumes matching those from the family of characteristic curves corresponding to the given regulated peak flow.)

Results

The results of our analysis include:

- Unregulated volume-frequency curves for Farmington Reservoir (as shown in Figure 2).
- Unregulated volume-frequency curves for Littlejohn Creek at Farmington, CA (as shown in Figure 3).
- Unregulated-regulated flow transform for Farmington Reservoir (as shown in Figure 4).
- Regulated flow-frequency curve and associated volumes for Farmington Reservoir (as shown in Table 1 and in Table 2).
- Unregulated-regulated flow transform for Littlejohn Creek at Farmington, CA (as shown in Figure 5).
- Regulated flow-frequency curve and associated volumes for Littlejohn Creek at Farmington, CA (as shown in Table 3 and in Table 4).
- Expected hydrograph properties for Farmington Reservoir. (Note: these are the same values shown in Table 1).

In addition, these intermediate values and information are included with the original report on DVD:

- HEC-DSS time series of the floods-of-records.
- HEC-DSS time series of the scaled historical floods.
- HEC-DSS time series of developed local flows below Farmington Reservoir (detailed in Attachment 2).
- The tabulated event maxima datasets for the 2 analysis sites.
- Simulated reservoir releases and routed flows from the HEC-ResSim reservoir simulation model.
- Tabulated unregulated-regulated flow transforms for the 2 analysis sites.
- Tabulated families of regulated characteristic curves for the 2 analysis sites.

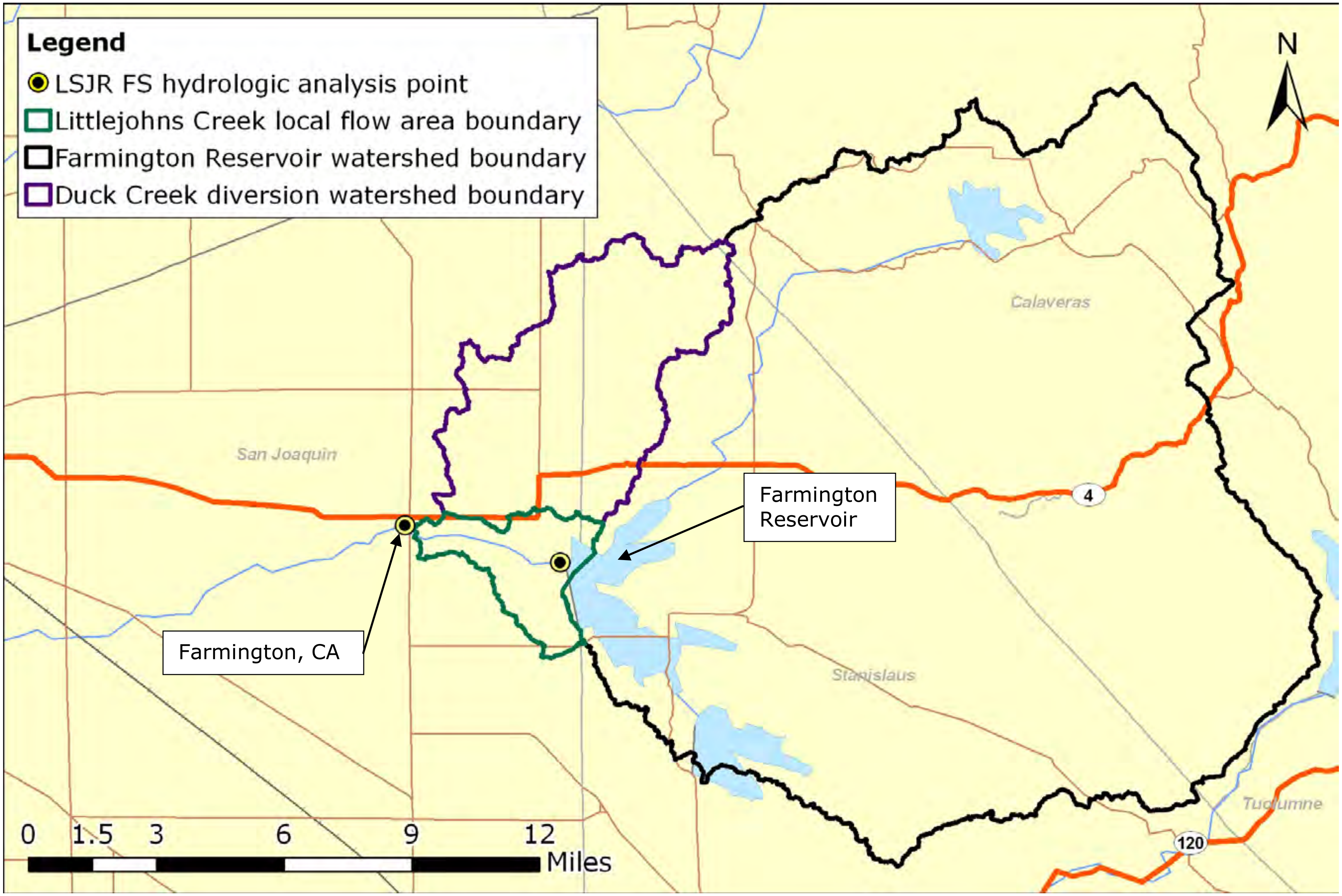
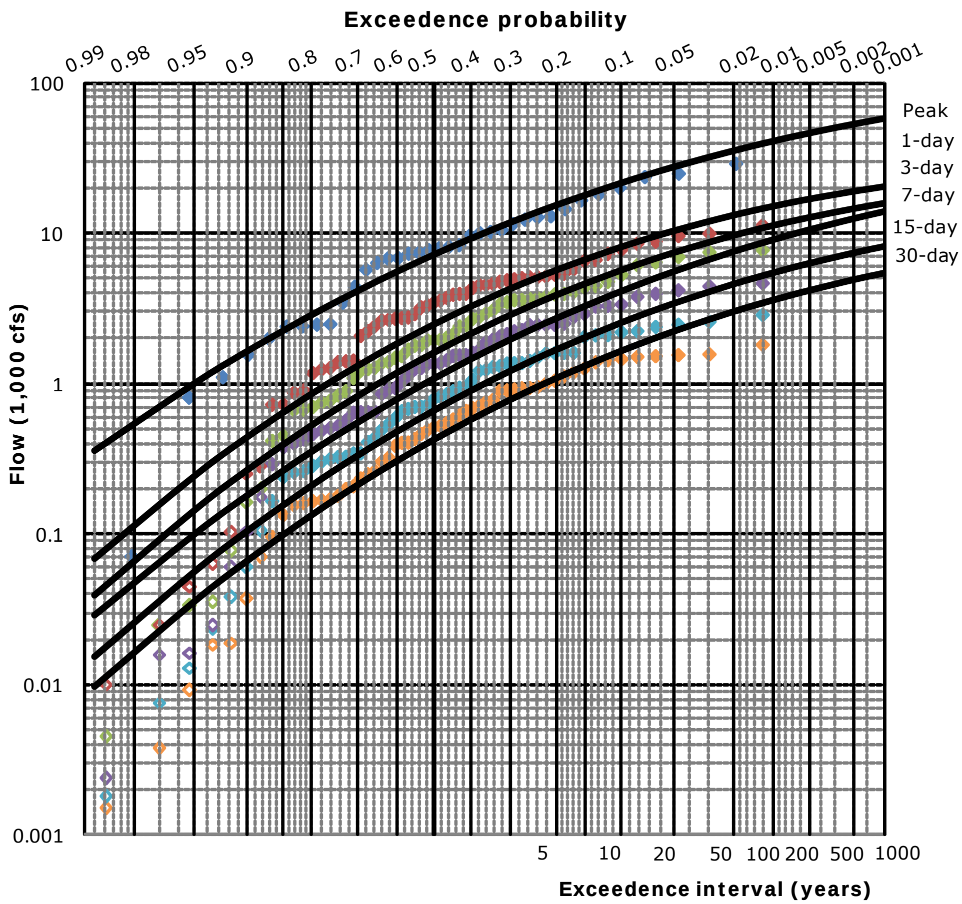


Figure 1. Littlejohn Creek study area



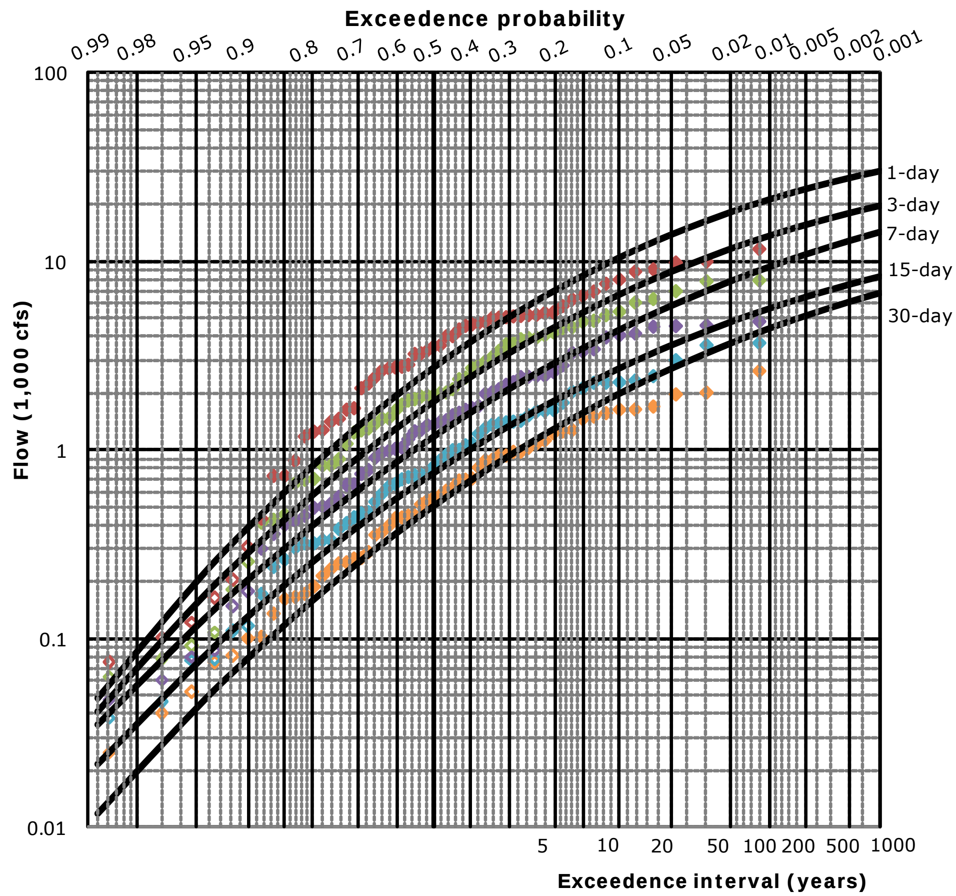
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.811	0.445	-0.692
1-day	3.321	0.507	-0.858
3-day	3.135	0.531	-0.812
7-day	2.970	0.538	-0.675
15-day	2.754	0.553	-0.733
30-day	2.561	0.556	-0.721

Notes:

- Median plotting positions.
- Drainage area: 212 sq. miles.
- Period of systematic record: 1951-2008.
  (Peak flow data intermittent 1952-2004).
- Record lengths
  Peak flows: 53 years.
  Volumes: 58 years.
- Regional skew values developed by USGS.
- Low outliers for volumes: 8 smallest events.
- Hollow points are censored events.

Figure 2. Unregulated frequency curves: Farmington Reservoir



Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
1-day	3.356	0.573	-0.849
3-day	3.186	0.545	-0.786
7-day	3.011	0.525	-0.670
15-day	2.815	0.523	-0.722
30-day	2.639	0.556	-0.695

Notes:

- Median plotting positions.
- Drainage area: 219 sq. miles.
- Period of systematic record: 1951-2008.
- Record length: 58 years.
- Regional skew values developed by USGS.
- Low outliers for 1-, 3, 7, and 15-day volumes: 7 smallest events.
- Low outliers for 30-day volumes: 6 smallest events.
- Hollow points are censored events.

Figure 3. Unregulated frequency curves: Littlejohn Creek at Farmington, CA

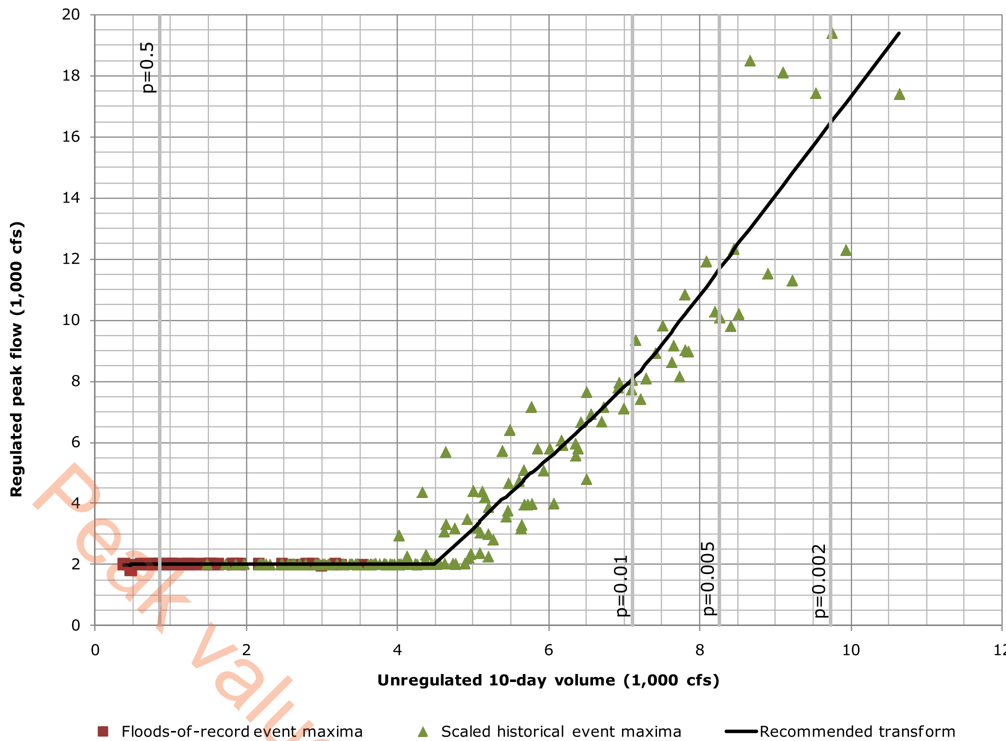


Figure 4. Unregulated-regulated flow transform: Farmington Reservoir

Table 1. Regulated peak flow-frequency quantiles: Farmington Reservoir

Annual exceedence probability (1)	**1/annual exceedence probability (2)**	**Regulated peak flow (cfs) (3)**
0.500	2	2,000
0.200	5	2,000
0.100	10	2,000
0.050	20	2,000
0.020	50	5,360
0.010	100	8,077
0.005	200	11,671
0.002	500	16,444

Table 2. Regulated peak flow values and associated volumes: Farmington Reservoir

Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes[1] (as average flow for given duration) 1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)	30-day (cfs) (7)
0.500	2,000	2,000	1,994	1,987	1,910	1,491
0.200	2,000	2,000	1,994	1,987	1,910	1,491
0.100	2,000	2,000	1,994	1,987	1,910	1,491
0.050	2,000	2,000	1,994	1,987	1,910	1,491
0.020	5,360	5,213	4,601	3,469	2,776	2,458
0.010	8,077	7,833	6,783	4,996	3,614	3,052
0.005	11,671	11,307	9,746	7,397	4,662	3,536
0.002	16,444	15,928	13,704	10,695	5,043	3,563

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at Farmington Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

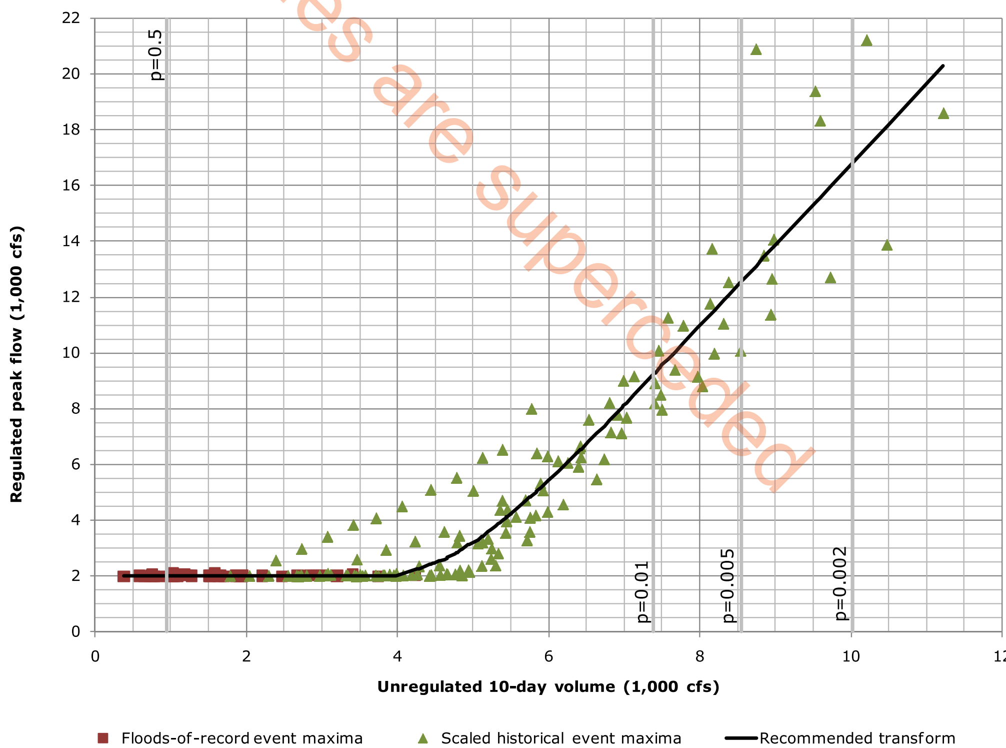


Figure 5. Unregulated-regulated flow transform: Littlejohn Creek at Farmington, CA

Table 3. Regulated peak flow-frequency quantiles: Littlejohn Creek at Farmington, CA

Annual exceedence probability (1)	1/annual exceedence probability (2)	Regulated peak flow (cfs) (3)
0.500	2	2,000
0.200	5	2,000
0.100	10	2,000
0.050	20	2,633
0.020	50	5,964
0.010	100	9,231
0.005	200	12,548
0.002	500	16,839

Table 4. Regulated peak flow values and associated volumes: Littlejohn Creek at Farmington, CA

Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes[1] (as average flow for given duration)				
		1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)	30-day (cfs) (7)
0.500	2,000	2,000	1,967	1,960	1,296	827
0.200	2,000	2,000	1,967	1,960	1,296	827
0.100	2,000	2,000	1,967	1,960	1,296	827
0.050	2,633	2,073	2,073	2,073	2,016	1,869
0.020	5,964	5,622	4,978	3,742	2,923	2,616
0.010	9,231	8,741	7,430	5,576	3,943	3,211
0.005	12,548	11,773	9,833	7,268	4,649	3,613
0.002	16,839	15,385	12,070	8,790	5,291	3,781

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at Farmington Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

Watershed description

The watershed that is the subject of this report—Littlejohn Creek basin—is part of the lower San Joaquin River basin. It is located in Calaveras, San Joaquin, and Stanislaus counties. Located on Littlejohn Creek approximately 20 miles upstream of Stockton, CA, is Farmington Reservoir, a "dry dam" whose primary purpose is flood control.

The principal feature of the watershed, shown in Figure 6, is Farmington Reservoir, which drains approximately 212 mi^2. The watershed above the reservoir is wing-shaped and extends 20 miles upstream into the foothills of the western Sierra Nevada. Elevations range from approximately 2,600 ft to approximately 115 ft at the dam.

In addition to runoff from the foothills, Farmington Reservoir receives flows from a diversion on the Stanislaus River at Goodwin Dam, the Stockton East Tunnel, and the Farmington-Stockton East Canal. These flows occur primarily during the summer months and not during the flood season, typically defined as October 1 to May 1 of each water year.

Downstream of Farmington Dam, approximately 3.5 miles, is the Duck Creek Diversion, which diverts flow into Littlejohn Creek from Duck Creek above the town of Farmington. The watershed above the diversion structure on Duck Creek is approximately 28 mi^2. The channel capacity of Duck Creek below the diversion structure is 700 cfs, and the diversion structure itself has a peak capacity of 500 cfs. In addition, the confluence of Littlejohn Creek and Rock Creek is approximately 2 miles downstream of Farmington Dam.

From the town of Farmington, Littlejohn Creek continues west, splitting into the North Fork Littlejohn Creek and South Fork Littlejohn Creek. Flow finally joins French Camp Slough before continuing on to the San Joaquin River. The confluence of Littlejohn Creek and French Camp Slough is located approximately 25 miles downstream of Farmington Dam.

Farmington Reservoir operates to maintain peak flows below the downstream channel capacity of 2,000 cfs near the town of Farmington, including anticipated coincident flows from the Duck Creek Diversion (USACE 2004).

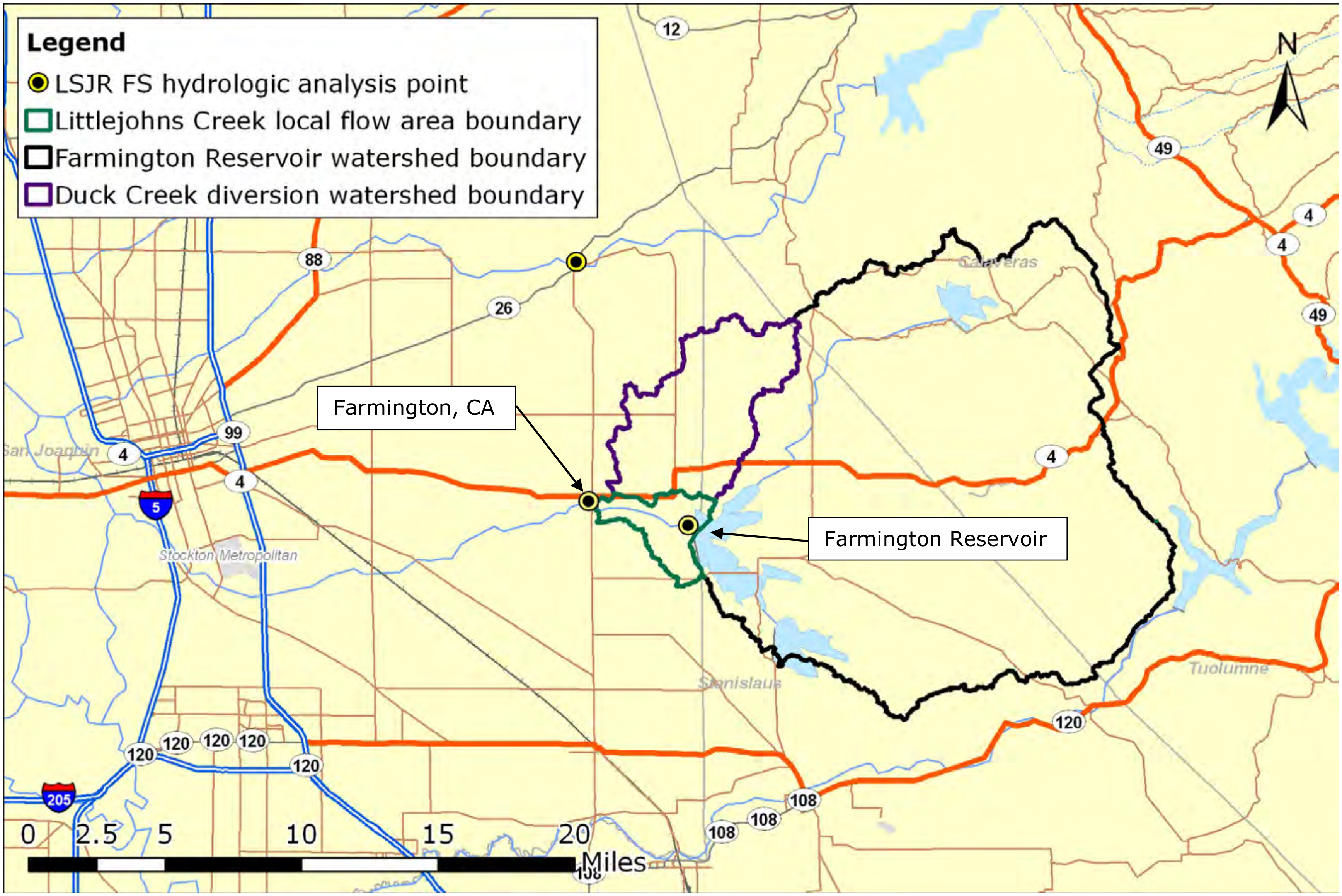


Figure 6. Lower San Joaquin River feasibility study area: Littlejohn Creek

Analysis procedure

Overview of CVHS procedure

The primary tasks for the CVHS are described in the *Procedures document*. More detail for these tasks is provided in the *Technical procedures document*. As a review of those tasks and to provide context for the procedures used in this analysis, here we summarize the procedure steps and categorize them into 2 groups. They are:

- Group 1. Unregulated frequency analysis at selected points. This comprises *Procedures document* Task 1, Task 2 (reservoir simulation models), Task 3, and Task 4. (References throughout this report to numbered tasks use numbers from the *Procedures document*.)
- Group 2. Assement of the effects of the regulation (flood control) system to convert the unregulated frequency curves to regulated flow-frequency curves at the same selected points. This comprises *Procedures document* Task 2 (channel routing models), Task 5, Task 6, and Task 7.

Group 1 focuses on completing a frequency analysis to characterize the annual exceedence probability of a given flow (unregulated). Thus, all statements of probability originate here.

Group 2 reflects the impact of regulation in the system. This second group accounts for various historical storm distributions and reservoir operations, with an emphasis on large events.

Application to the lower San Joaquin River feasibility study

In Figure 7, we illustrate the general work flow of the analysis procedure as applied to the LSJR FS. In this document we note before each analysis step the corresponding CVHS procedures task applicable, if any.

For unregulated frequency analysis for the 2 sites on Littlejohn Creek, Farmington Reservoir and Farmington, CA, we:

- (Task 1) Obtained reservoir inflow and streamgage data for use in developing the unregulated flow time series from the Corps.
- (Task 2) Obtained accepted reservoir simulation and channel routing models from the Corps.
- (Task 3) Developed unregulated flow time series at each location corresponding to a period-of-record of floods. This step includes the development of local flows for the ungaged area between New Hogan Dam and Farmington, CA.
- (Task 4) Computed and adopted unregulated 1-, 3-, 7-, 15-, and 30-day volume-frequency curves at each location. Note: we developed peak unregulated flow-frequency curves for Farmington Reservoir for completeness; they are not required for this analysis.

For regulated system analysis for the 2 sites on Littlejohn Creek we:

- (Task 5) Developed regulated flow time series at each location by simulating and routing reservoir releases. Here, historical and scaled historical events were used in development of the time series.

- (Task 6) Fitted flow transforms. First, the unregulated and corresponding regulated event maxima datasets were identified (these are data points to which the transforms were fitted). Then, the critical duration of each analysis location was determined using these series. The flow transforms were then developed by fitting curves to the event maxima datasets. Note here, the term flow transforms refers to: (1) the unregulated-regulated flow transform, and (2) the family of regulated characteristic curves.
- (Task 6.4) Applied flow transforms to develop a regulated peak flow-frequency curve and associate volumes for the 1-, 3-, 7-, 15-, and 30-day durations at each location.

For development of the expected hydrograph properties for Farmington Reservoir outflows we identified the peak regulated flows and associated regulated volume-duration characteristics for 8 exceedence probabilities: p=0.5, p=0.2, p=0.1, p=0.05, p=0.02, p=0.01, p=0.005, and p=0.002.

Attachment 1 provides a table explaining how the procedures detailed here align with the procedural steps detailed in the *Procedures document* and the *Technical procedures document*.

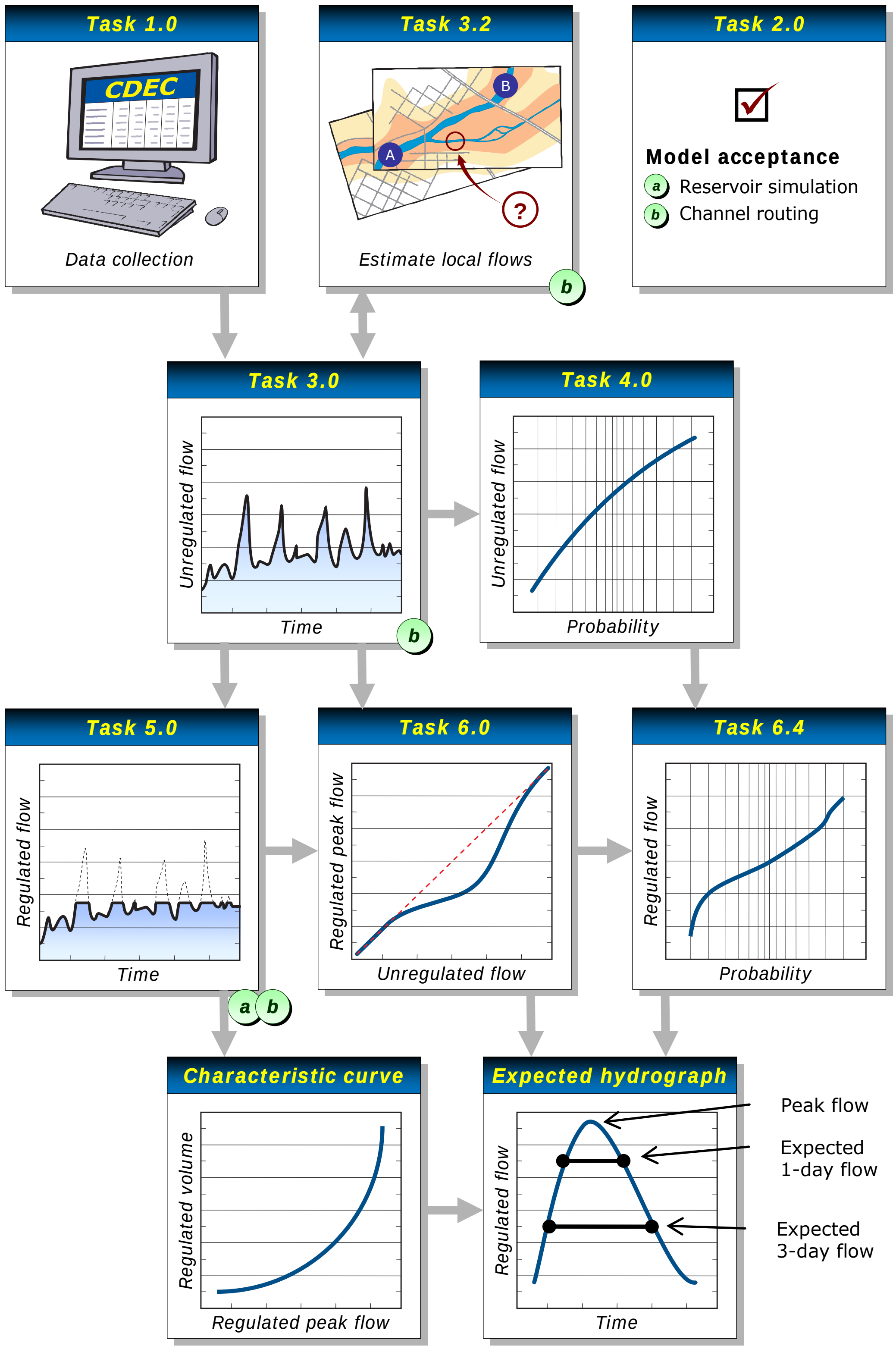


Figure 7. LSJR analysis procedure workflow

Unregulated flow time series development

We constructed unregulated flow time series at each analysis location in the study area and fitted unregulated volume-frequency curves to these series using procedures that are consistent with Corps guidance.

The locations most upstream at which we developed unregulated flow time series were the project reservoirs. Thus, for unregulated conditions, the reservoir inflows were needed.

For development of the unregulated flow time series downstream of the reservoir, a routing model was required to simulate the translation, attenuation, and combination of the unregulated flow hydrographs through the system. These flow hydrographs included the upstream boundary conditions (derived reservoir inflows) and intermediate area boundary conditions (estimated local flows). The routing yielded unregulated flow time series that served as the basis of: (1) the unregulated frequency analysis and (2) the unregulated-regulated flow transform.

For this analysis, we developed an unregulated flow time series for the 2 analysis locations on Littlejohn Creek by:

- (Task 1) Obtaining daily unregulated reservoir inflow time series developed by the Corps.
- (Task 3.2) Developing local flow time series for the area between Farmington Reservoir and the reservoir's control point at Farmington, CA (shown in Figure 8).
- (Task 3.3) Completing the unregulated flow time series at each analysis point.

Obtain daily reservoir inflow

We obtained the daily unregulated reservoir inflows from the Corps. The Corps developed the daily unregulated reservoir inflow time series for Farmington Reservoir using the continuity equation, in which, for a given time step, the average inflow equals the outflow plus the change in reservoir storage. For the calculation of these inflows, the source of the observed reservoir outflows and observed changes in storage was the Corps's database. By convention in the Central Valley, these calculations were completed on a 1-day time step, thus midnight to midnight values were used. This is consistent with the work completed for the *Sacramento and San Joaquin river basins comprehensive study* (Comp Study) completed in 2002 (USACE 2002).

Estimate local flow

For Littlejohn Creek, local flows needed to be estimated for the area between Farmington Reservoir and Farmington, CA, shown in Figure 8. The estimation approaches we used were:

- Option 1. Direct calculation of local flow using known releases from Farmington Reservoir, known diversions from Duck Creek, and the observed flows at Farmington, CA, routing hourly flows as necessary. In the case of missing streamgage data, local flows values were interpolated as needed.

- Option 2. Estimation of local flows as:

$$Q_{Local} = 0.04\left(Q_{FRM}\right) \tag{1}$$

where Q_{Local} is the local flow estimate for a given time, and Q_{FRM} is the unregulated inflow to Farmington Reservoir. The Corps estimates local flows for the purpose of real-time reservoir operations using this option (John High, personal communication, 11/9/2009) and this is the option used to estimate local flows in the Comp Study (USACE 2002).

In Table 5 we summarize the selected approaches for local flow estimation on Littlejohn Creek by water year. This flow represents the total local flow contribution at Farmington, CA. We detail the development of the local flow time series on Littlejohn Creek in Attachment 2.

Table 5. Selected local flow estimation approaches for the area on Littlejohn Creek between Farmington Reservoir and Farmington, CA

Time period (water year) (1)	Time step (2)	Selected approach[1] (3)
1951-1968	Daily	Option 1: directly calculate local flow.
1969-1970	Daily	Option 2: 0.04 times reservoir inflow.
1971-1972	Daily	Option 1: directly calculate local flow.
1973	Daily	Option 2: 0.04 times reservoir inflow.
1974-1996	Daily	Option 1: directly calculate local flow.
1996-2008	Hourly	Option 1: directly calculate local flow.

1. The approach listed is the predominant method for estimating local flows over the time period given. See Attachment 2 for further detail.

Complete unregulated flow time series

For the unregulated frequency analysis, we used the daily unregulated reservoir inflow time series provided by the Corps directly as the unregulated time series corresponding to Farmington Reservoir. For the reservoir's operation point on Littlejohn Creek at Farmington, CA, we combined the daily unregulated inflow time series with the estimated local flows by adding the 2 time series together. We did not route the unregulated reservoir inflows because: (1) synthesizing a shorter time step is not required for frequency analysis, and (2) the travel time between the reservoir and the operation point is approximately 2 hours, which is less than the 1-day time step of the inflows. In addition, there is little attenuation of flood peaks in this reach because of its length and channel geometry. We confirmed this by comparing observed releases from Farmington Reservoir, observed diversions from Duck Creek, and observed flows on Littlejohn Creek at Farmington, CA. The unregulated flow time series at Farmington, CA, does not include diversions from Duck Creek.

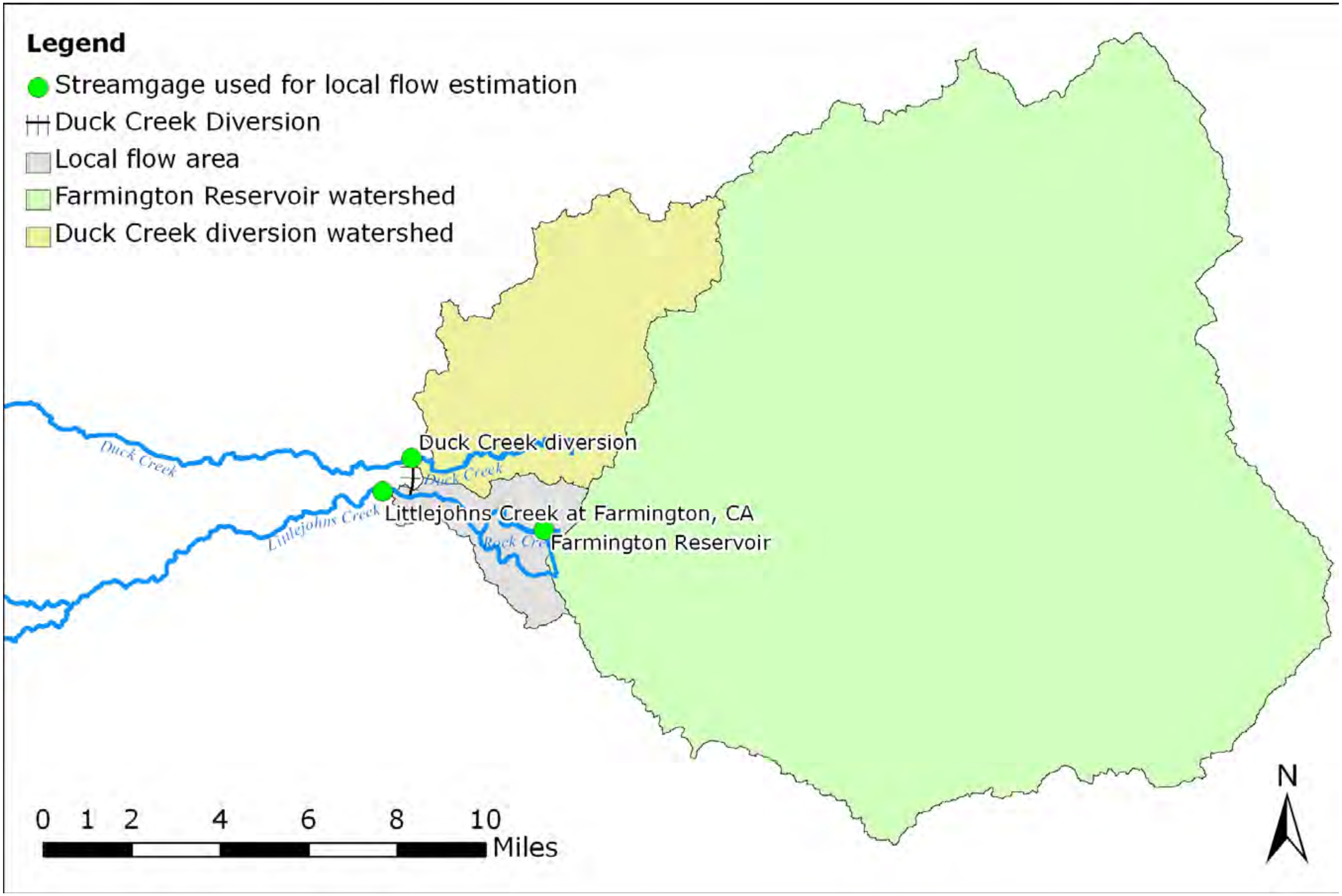


Figure 8. Littlejohn Creek local flow area between Farmington Reservoir and Farmington, CA, and study streamgages

Unregulated frequency analysis

Commonly accepted procedures to develop unregulated flow-frequency curves are specified in *Bulletin 17B* (IACWD 1982). The current standard-of-practice is to fit a Pearson III (LPIII) distribution to the logarithmic transforms of annual maximum series identified from streamgage data. Additional guidance for fitting frequency curves to volumes for a given duration is provided by EM 1110-2-1415 (USACE 1993).

For this analysis, the unregulated inflows to Farmington Reservoir can be used to develop such an annual maximum series. However, because we only had records of regulated flows on Littlejohn Creek at Farmington, CA, we could not fit a frequency curve directly using this method. Thus, we used the synthesized unregulated flow time series at this location and fitted a volume-frequency curve to that series using procedures that are consistent with Corps guidance.

For this analysis we developed unregulated frequency curves following the procedures specified in *Bulletin 17B* (IACWD 1982), EM 1110-2-1415 (USACE 1993), and the current standards of practice. For each analysis location, we:

- Identified the annual maximum series.
- (Task 4.1) Calculated regional skew values for each duration of interest using relationships developed by the USGS.
- (Task 4.2) Fitted LPIII distributions to the annual maximum series following *Bulletin 17B* procedures and Corps guidance using the expected moment algorithm (EMA) enabled flow-frequency software PeakfqSA, version 0.937. This was developed by Tim Cohn of the USGS and is based on the USGS's flow-frequency software PeakFQ (Cohn 2007).
- Reviewed and adopted the curves, checking them for consistency and comparing them to previously accepted values.

Identify annual maximum series

We identified the annual maximum series by extracting, from the unregulated flow time series, the volumes associated with the 1-, 3-, 7-, 15-, and 30-day durations. This information is detailed in Attachment 3.

We developed a peak unregulated flow-frequency curve for Farmington Reservoir for completeness; however this is not required for this analysis. The peak annual maximum series was provided by the Corps and is included in Attachment 3. In addition, we did not develop a peak flow-frequency curve for Littlejohn Creek at Farmington, CA, because the temporal resolution of the unregulated flow time series, 1 hour to as long as 1 day, is not an appropriate representation of instantaneous unregulated peak flow values.

Calculate regional skew values

For this analysis, we calculated regional skew values for the peak flows and 1-, 3-, 7-, 15-, and 30-day volumes using the relationships developed by the USGS (USGS 2010). In these relationships, the regional skew value is a function of the average basin elevation. The values calculated for each analysis location and duration of interest are shown in Attachment 4.

Fit frequency curves

To fit frequency curves to the annual maximum series we used: (1) the statistics of the logarithmic transforms of unregulated flow time series (mean, standard deviation, and skew), and (2) the regional skew values for the peak flow, and 1-, 3-, 7-, 15-, and 30-day calculated using relationships developed by the USGS (2010). The "at station" statistics were calculated using the EMA option in PeakfqSA.

As a first step, the curves were fitted using a straightforward *Bulletin 17B* procedure in which all data points were included in the analysis and low outliers were identified by the *Bulletin 17B* outlier test (implemented automatically by the program). The station statistics were then appropriately adjusted. This includes weighting the station skew and regional skew values by the inverse of their associated errors. This weighting procedure is included in *Bulletin 17B*, and the weighted skew is automatically calculated by PeakfqSA.

We found that this initial fitting of the frequency curves: (1) was sensitive to low flow values, and (2) the 1-day and 3-day flow quantiles for p= 0.01 and p=0.005 annual exceedence probabilities were uncharacteristically large on a flow-per-square mile basis.

We then refitted the frequency curves setting the low outlier thresholds for each duration. Specifically, we set these thresholds consistent with those used in the Comp Study. In addition, we adjusted the standard deviations, following guidance in EM 1110-2-1415 (USACE 1993), for consistency. This fitting is detailed Attachment 4.

Review and adopt curves

After fitting, we reviewed the frequency curves for consistency and appropriateness. Specifically, we:

- Compared the curve of a given duration to the curves associated with the other durations at the same analysis location.
- Compared the curves at a given location to the curves at the other analysis location to check for consistency.
- Compared the curves to those published in the Comp Study.

We found the frequency curves on Littlejohn Creek were consistent between durations at each location for the frequencies of interest. The curves do not "cross," and flow quantiles for a given duration at the downstream location are greater than those of the upstream location, as would be expected.

As a comparison, we considered the volume-frequency curves developed for Farmington Reservoir in the Comp Study (USACE 2002). The annual maximum series in the Comp Study ended in 1998.

We also found that compared to the flow quantiles in the Comp Study the quantiles of the curves fitted here are: (1) smaller for the 1 day duration, and (2) larger for durations equal 3-days or greater. (Here the only exception is the 3-day p=0.5 quantile which we found to be approximately 9% less than that of the Comp Study.) However, we found that the 1-day and 3-day flow quantiles for p=0.01 and p=0.005 annual exceedence probabilities were consistent with those from nearby watersheds on a flow-per-square mile basis. In this analysis, the peak flow-frequency quantiles varied by as much as 9%, as compared to those in the Comp Study, because of (1) the additional 6 events include, 1999 through 2004, and (2) the use of EMA in fitting the curve.

We adopted the unregulated frequency curves for the 2 analysis locations, Farmington Reservoir and Farmington, CA, shown in Figure 9 and Figure 10. These are the curves that use manually specified low outlier thresholds. The detailed parameters used to fit these curves are included in Attachment 4.

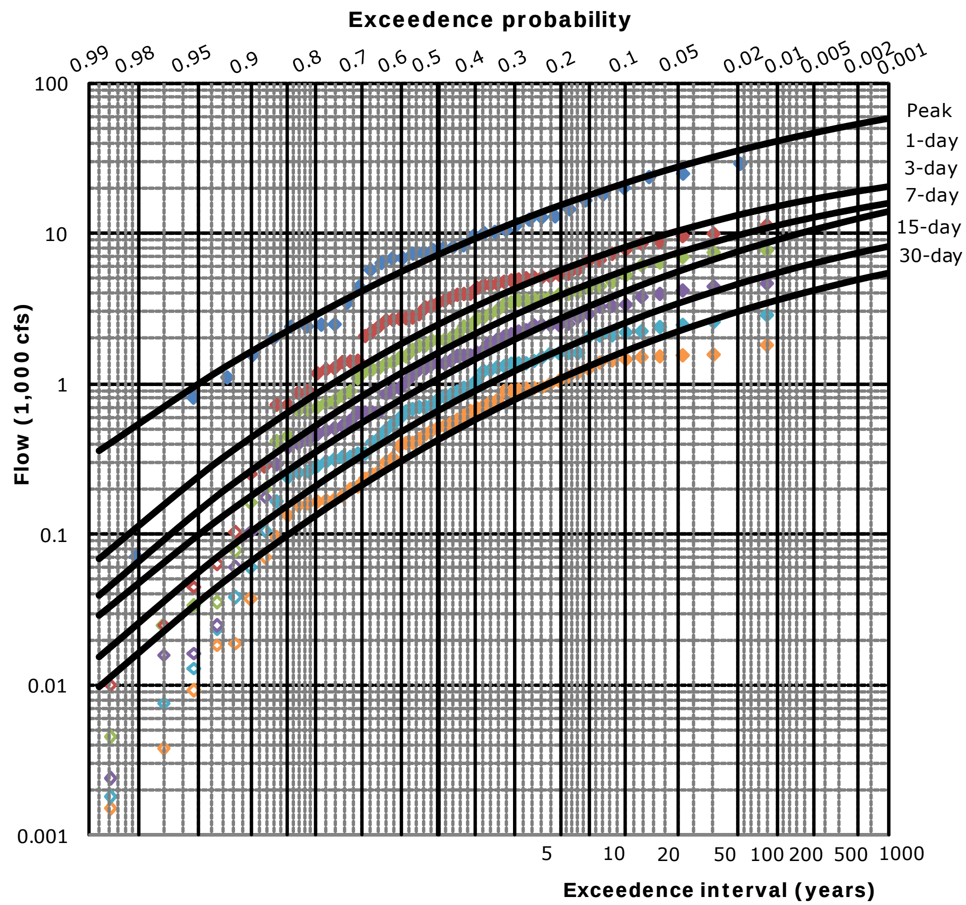


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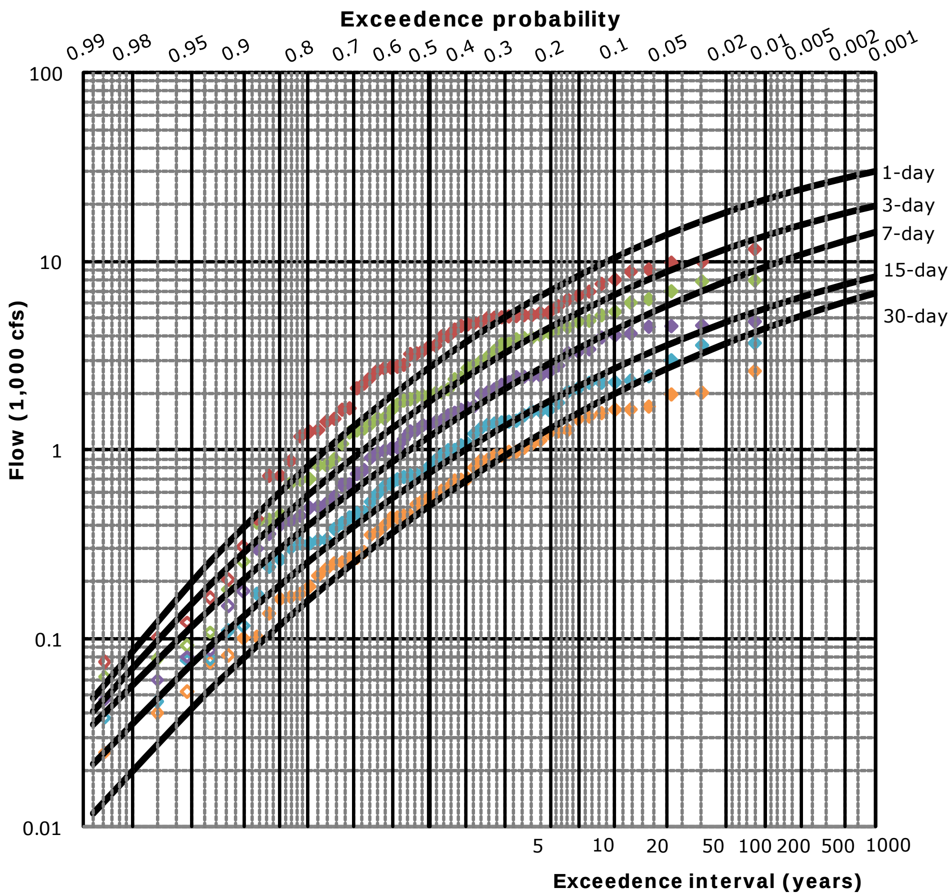
Figure 9. Unregulated frequency curves: Farmington Reservoir



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3-day	3.186	0.545	-0.786
7-day	3.011	0.525	-0.670
15-day	2.815	0.523	-0.722
30-day	2.639	0.556	-0.695

Notes:

- Median plotting positions.
- Drainage area: 219 sq. miles.
- Period of systematic record: 1951-2008.
- Record length: 58 years.
- Regional skew values developed by USGS.
- Low outliers for 1-, 3, 7, and 15-day volumes: 7 smallest events.
- Low outliers for 30-day volumes: 6 smallest events.
- Hollow points are censored events.

Figure 10. Unregulated frequency curves: Littlejohn Creek at Farmington, CA

Regulated flow time series development

To develop regulated flow-frequency curves, the unregulated volume-duration-frequency curves are transformed through the unregulated-regulated flow transform. The unregulated-regulated flow transform captures the system's response to large, varied events, and is created using the unregulated and regulated flow time series. To develop the regulated flow time series we took selected historical events from the unregulated flow time series and simulated those in the regulated system. In addition, scaled historical events were used to represent events larger than those seen in the historical record for definition of the flow transforms. We then compiled the maximum unregulated and regulated flows for various durations to develop the event maxima datasets.

For this analysis we developed the regulated flow time series at each analysis location by:

- Smoothing the unregulated flow time series, using those series as boundary conditions to the reservoir simulation model.
- Identifying floods-of-record (discrete events) required to develop the flow transforms.
- Scaling historical events to represent events larger than those in the historical record.
- (Task 5.1 and Task 5.2) Simulating and routing reservoir releases of historical and scaled events.

Smooth unregulated flow time series

The daily unregulated flow time series are appropriate for frequency analysis. However daily upstream and intermediate boundary conditions do not have the temporal resolution required by the CVHS procedures for assessing the effects of regulation, particularly releases as indicated on the emergency spillway release diagram (ESRD). Therefore, the daily reservoir inflows and daily estimated local flows were "smoothed" to hourly time series. This smoothing was completed using a mass balance algorithm that interpolates the shape of the hydrograph and estimates peak hourly flows while maintaining daily volumes consistent with the original time series. These smoothed times series were provided by the Sacramento District Hydrology Section for use in this analysis.

Identify floods-of-record

Events rarer than p=0.5 annual exceedence event are needed to define the flow transforms. To develop the flow transforms we used both historical events and scaled historical events. The 40 historical events used were those with 1-day volumes greater than 2,000 cfs (a threshold slightly lower than volume corresponding to the p=0.5 exceedence event.)

To select the subset of events used for scaling, we identified: (1) the 14 large flood events for the San Joaquin River basin (listed in the Comp Study historical storm matrices), and (2) the 5 largest events for Littlejohn Creek watershed (of which only the 2006 event was not included in the Comp Study matrices). We list these events in Table 6. In Table 6, column 1 lists the water year of the event, column 2 and column 3 list the associated start and end dates, column 4 lists the 1-day volume, and column 5 indicates the selection basis. On Littlejohn Creek, 4 of the 5 largest inflow events are included in the Comp Study historical storm matrix. We identified these dates by visual inspection of unregulated inflow time series provided by the Corps. The time windows defined by these dates was used for extraction of the event maxima (unregulated and regulated) for development of the flow transforms.

The Comp Study lists both a January and February event for the 1969 water year in the San Joaquin River basin. However, a large February inflow event is not present in the Farmington Reservoir unregulated inflow time series. Therefore, for this analysis we treat the 1969 flood as a single event.

Table 6. Littlejohn Creek floods-of-record scaled to develop flow transforms

Water year[1] (1)	**Start date (2)**	**End date (3)**	**1-day max volume (cfs) (4)**	**Selection basis (5)**
1998	1/26/1998	2/28/1998	11,270	Comp Study storm matrix event
2006	3/26/2006	4/30/2006	9,912	Largest inflow event
1986	1/26/1986	2/28/1986	9,555	Comp Study storm matrix event
1965	12/20/1964	1/20/1965	8,760	Comp Study storm matrix event
1956	12/20/1955	2/5/1956	8,497	Comp Study storm matrix event
1997	12/28/1996	2/12/1997	7,777	Comp Study storm matrix event
1958	3/12/1958	4/12/1958	7,272	Comp Study storm matrix event
1983	11/20/1982	3/31/1983	6,620	Comp Study storm matrix event
1982	12/27/1981	4/20/1982	6,522	Comp Study storm matrix event
1951	11/17/1950	12/31/1950	5,284	Comp Study storm matrix event
1980	1/10/1980	3/10/1980	4,921	Comp Study storm matrix event
1995	1/1/1995	3/31/1995	4,854	Comp Study storm matrix event
1967	1/20/1967	4/30/1967	4,324	Comp Study storm matrix event
1969[2]	1/10/1969	3/10/1969	3,707	Comp Study storm matrix event
1978	1/4/1978	3/20/1978	3,447	Comp Study storm matrix event

1. Events are in order of increasing 1-day flow volume
2. For the purposes of this analysis we treat the 1969 flood as 1 single event.

Scale historical floods

In addition to the 40 historical floods-of-record, events larger than these recorded were required to develop the flow transforms throughout the full range of interest. To obtain those, we scaled the time series for the subset of historical event s listed in Table 6 uniformly by factors at 0.2 intervals from 1.2 through 3.0 for use in simulating reservoir releases. This yielded a total of 10 scaled time series for each event. Both the unregulated reservoir inflow and estimated local flow time series were scaled uniformly to maintain the coincidence and timing of the system.

Scaled historical events were used only for the development of the flow transforms. The events were not used for fitting the unregulated flow frequency curves. This use of scaled historical events is consistent with the guidance in EM 1110-2-1415.

Simulate and route historical and scaled floods

We simulated reservoir operation and routed flows for both the historical floods-of-record and scaled historical events using the computer program HEC-ResSim, version 3.1 Beta III, developed by the USACE Hydrologic Engineering Center (HEC). Given a reservoir network, operating rules and constraints, and a set of inflows and downstream local flows, HEC-ResSim routes the flows through the system and simulates releases for the reservoirs. These releases are based on the rules and constraints defined in the water control manual.

An HEC-ResSim reservoir network includes representation of the physical properties of the reservoirs and links from reservoirs to downstream points of interest. Hydrologic routing model parameters are required to represent the movement of the flood wave between nodes in the network. Required physical properties include elevation-volume relationships, elevation-maximum outflow relationships, and physical limitations of the reservoir outlets.

The operating rules defined for a reservoir for HEC-ResSim include release functions based on reservoir pool elevation, reservoir inflow, and downstream flow constraints. Rate of change constraints are also included in the operation rule sets. For Littlejohn Creek, Farmington Reservoir operates to meet downstream flow constraints at Farmington, CA, which is just below the inflow from the Duck Creek diversion, approximately 3.5 miles downstream of the reservoir.

Simulate reservoir operation

For this analysis, we used the representation of the Littlejohn Creek system in HEC-ResSim developed by the Corps; that will be used for the CVHS. This includes a representation of the network and the reservoir operation rules. The HEC-ResSim schematic of the Littlejohn Creek system is shown in Figure 11. The major features of the network shown in Figure 11 are: Farmington Reservoir, the diversion from Duck Creek, and the reservoir control point at Farmington, CA.

For reference, Farmington Reservoir is operated to maintain flows in Littlejohn Creek at Farmington, CA, below 2,000 cfs. The complete set of operating rules is defined in the Farmington Reservoir water control manual (USACE 2004).

With this model, we simulated the 15 historical floods-of-record and associated scaled events for a total of 165 simulations. Consistent with the standard-of-practice for such analysis, for the reservoir routings, we used only the dedicated flood control storage space for the attenuation of the reservoir inflows. Thus, at the start of the simulation, the reservoir water surface elevation equals the elevation of the bottom of the flood control pool. The simulation time step for this analysis is 1 hour.

After completing the reservoir simulations, we reviewed the results from the HEC-ResSim computer program. We found that the simulated releases were consistent with our knowledge of the system operation and water control manual.

Route reservoir releases

We used Muskingum routing to route flows on Littlejohn Creek. A detailed channel model of Littlejohn Creek does not currently exist. Although the *Procedures document* calls for the hydraulic routing of reservoir releases, we found that Littlejohn Creek can be adequately simulated with hydrologic routing because: (1) the analysis locations on Littlejohn Creek are not affected by backwater and therefore do not require evaluation of stages to develop regulated flow-frequency curves, and (2) the reservoir release hydrographs do not rise quickly. The results from the reservoir simulation and routing are provided on a DVD with the original report.

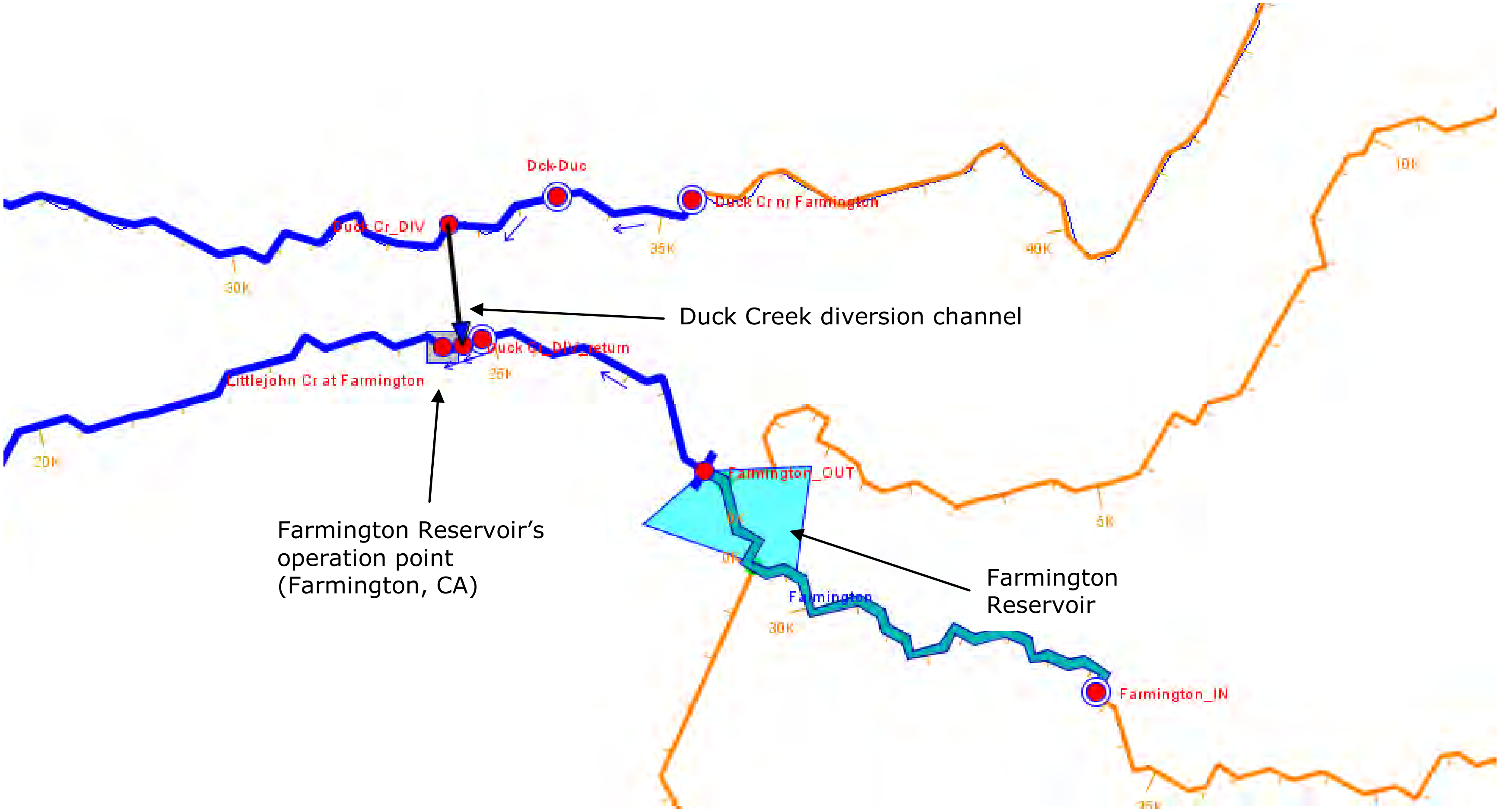

Figure 11. Screenshot of HEC-ResSim system schematic: Littlejohn Creek system

Flow transform fitting and application

Once the regulated flow time series were developed, the next step was to pair, by event, the unregulated and regulated flow time series. Using these pairings, the event properties, such as the volumes for given durations, and in the case of the regulated time series, peak flows, were identified. The result of this pairing and identification was the event maxima dataset. Specifically, the event maxima dataset consists of unregulated and regulated flows of various durations for a given historical or scaled historical event.

Once the event maxima datasets were compiled, a transform curve was fitted to develop the unregulated-regulated flow transforms. This curve translated the unregulated flow of a given quantile to the corresponding regulated flow for that same quantile. This process is illustrated in Figure 12.

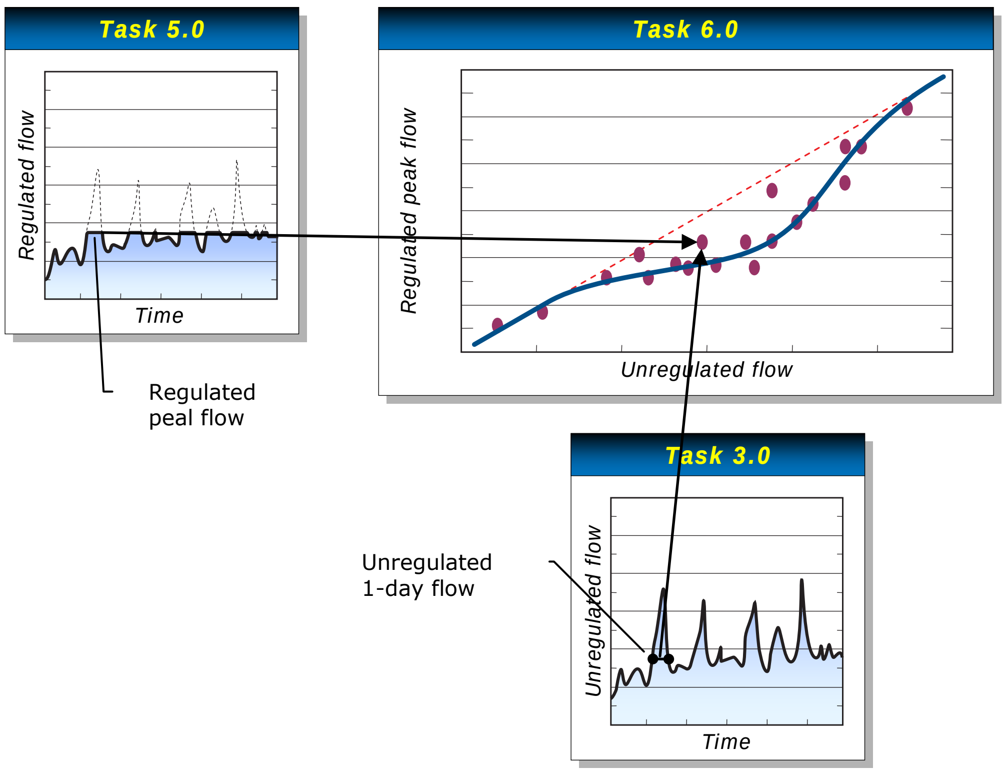

Figure 12. Flow transform development process

For the unregulated-regulated flow transform, the regulated flow value used was the peak flow. The unregulated flow value was the unregulated flow corresponding to the critical duration for that analysis location. The critical duration was found through an analysis of unregulated and regulated flows for historical and scaled historical events.

Additional transform curves were fitted to develop the family of characteristic curves. These curves identified the associated regulated volume duration characteristics of a given peak regulated flow.

For this analysis, we developed the flow transforms by:

- (Task 6.1) Identifying unregulated and regulated event maxima for the floods-of-record.
- (Task 6.2) Fitting the unregulated-regulated flow transform for each duration of interest.
- Determining the critical duration to identify the appropriate unregulated-regulated transform to use at each analysis location.
- Fitting the family of characteristic curves.
- Reviewing and accepting the flow transforms.

We then applied the flow transforms to the unregulated frequency curves to develop the regulated flow-frequency curves (Task 6.4).

Identify event maxima datasets

We identified the event maxima datasets using inspection and HEC-DSS utilities. For each analysis location, we:

- Identified the properties of the 1-, 1.5-, 2-, 2.5-, 3-, 3.5-, 4-, 4.5-, 5-, 6-, 7-, 10-, 15-, and 30-day durations for unregulated flows associated with the floods-of-record. The durations we used are consistent with those specified in the *Technical procedures document* for analyzing critical duration.
- Identified the peak regulated flows from the regulated flow time series of the historical floods-of-record and scaled historical events. Note that here, peak regulated flow corresponds to the maximum hourly value regulated flow time series, and not a true instantaneous peak.
- Identified the properties of the 1-, 3-, 7-, 15-, and 30-day durations for regulated flows associated with the historical floods-of-record and scaled historical events. We did not include all the durations used in the critical duration analysis consistent with those specified in the *Technical procedures document* and the current standard-of-practice for flow-frequency analysis.

The event maxima datasets are tabulated in an MS Excel file on a DVD provided with the original report. The tabulated information lists each historical and scaled historical event used in this analysis and the associated volumes for the (1) unregulated flow volumes corresponding to the 1-, 1.5-, 2-, 2.5-, 3-, 3.5-, 4-, 4.5-, 5-, 6-, 7-, 10-, 15-, and 30-day durations, and (2) regulated flow volumes corresponding to the peak, 1-, 3-, 7-, 15-, and 30-day durations.

Note that the unregulated event maxima do not include diversions from Duck Creek, while the regulated event maxima include diversions from Duck Creek.

Fit unregulated-regulated flow transforms

We developed the unregulated-regulated flow transforms for the 2 analysis locations by fitting transform curves through the pairs of event unregulated volumes and regulated peak flows. The unregulated volumes used were the average flows associated with the durations previously noted. We fitted these curves to the data pairs of historical and scaled events using the robust locally weighted scatterplot smoothing (LOWESS) regression technique. (The LOWESS procedure is detailed in the *Technical procedure document*.)

Here, we fitted these transforms for the 1-, 1.5-, 2-, 2.5-, 3-, 3.5-, 4-, 4.5-, 5-, 6-, 7-, 10-, 15-, and 30-day durations. The event maxima datasets include both historical and scaled events to define the extreme end of the flow transform curves. Fitting of the transforms are detailed in Attachment 5.

The CVHS analysis procedure requires 1 single unregulated-regulated transform for statements of probability. To identify which duration is most appropriate, the critical duration for the given analysis location must be determined as described in the next subsection.

Determine critical duration

We determined critical duration at each analysis location by: (1) applying the unregulated-regulated flow transforms to the unregulated flow–frequency curves to develop hypothetical regulated flow-frequency curves, and (2) identifying the duration of the unregulated annual maximum series that consistently estimates the largest flow for each probability. In selecting the critical duration, we considered both the "goodness of fit" of each transform and which duration estimates the greater peak regulated flows. This procedure is described in more detail in Attachment 5.

From this analysis we determined that the critical duration at Farmington Reservoir and at Farmington, CA, is 10 days. Thus, the appropriate unregulated-regulated flow transforms used in this analysis were associated with this duration. The critical duration associated with the downstream operation point is shorter than that of the reservoir because of the effects of local flow.

After determining the critical duration associated with each analysis location, we reviewed and adjusted the unregulated-regulated flow transforms initially fitted with the LOWESS procedure as detailed in Attachment 5. We then adopted the flow transforms for Farmington Reservoir and Farmington, CA, shown in Figure 13 and Figure 15. In Figure 13 and Figure 15, some scaled historical event maxima for more common events have regulated peaks exceeding the channel capacity (2,000 cfs) because of large local flows and diversions from Duck Creek.

Fit family of regulated characteristic curves

We developed the families of regulated characteristic curves for Farmington Reservoir and at Farmington, CA, by fitting most likely curves through the pairs of event regulated volumes as average flows and regulated peak flows, similar to the procedure we used to fit the unregulated-regulated transforms. The data pairs (from the event maxima datasets) we used include both historical and scaled events to define the extreme ends of the flow transform curve.

The family of regulated characteristic curves for Farmington Reservoir and Farmington, CA, are shown in Figure 14 and Figure 16, and are detailed in Attachment 6. These curves associate regulated peak flows to regulated characteristic volumes. We fitted characteristic curves for the 1-, 3-, 7-, 15-, and 30-day durations. We compare these families of curves in Figure 17.

On Littlejohn Creek, the typical duration of releases from Farmington Reservoir for events in the given range of interest is less than 15 days.

Therefore we include the 15-day and 30-day characteristic curves here for completeness, and in keeping with the CVHS procedures.

Review and adopt flow transforms

After fitting the flow transforms and characteristic curves, we reviewed the resulting functions for consistency. Specifically, we compared each transform to (1) the transforms associated with different durations at the same analysis location, and (2) the transforms at the other analysis location. We found:

- The unregulated-regulated flow transforms were consistent between analysis location, i.e., the regulated peak flow for a given quantile at the downstream location was greater than that of the upstream location.
- At both analysis locations, the families of regulated characteristic curves were consistent between durations, i.e., they do not cross. This is expected.
- The fit of the curves at Farmington, CA, was sensitive to large diversions from Duck Creek such as those in the 1995 event and its corresponding scaled events. For scaled versions of this event, the diverted exceeded channel capacity before the Farmington Reservoir flood control pool was filled.

Based on this review, we adopted these flow transforms for the 2 analysis locations.

Apply flow transforms

We developed a regulated peak flow-frequency curve and the associated regulated 1-, 3-, 7-, 15-, and 30-day volumes at Farmington Reservoir and at Farmington, CA, by combining the appropriate information from the unregulated frequency curves, the flow transforms, and the families of regulated characteristic curves. The regulated flow-frequency curves for Farmington Reservoir and Farmington, CA, are shown in Table 7 and Table 9 and their associated volumes are tabulated in Table 8 and Table 10.

To apply the flow transforms and develop regulated flow-frequency curve associated volumes at each analysis location we:

- Identified the unregulated flow quantiles associated with the critical duration that correspond to the probabilities of interest.
- Identified the regulated peak flows that correspond to the flow quantiles identified in the previous step using the flow transform.
- Identified the regulated flow characteristics that correspond to the regulated peaks identified in the previous step using the family of regulated characteristic curves.

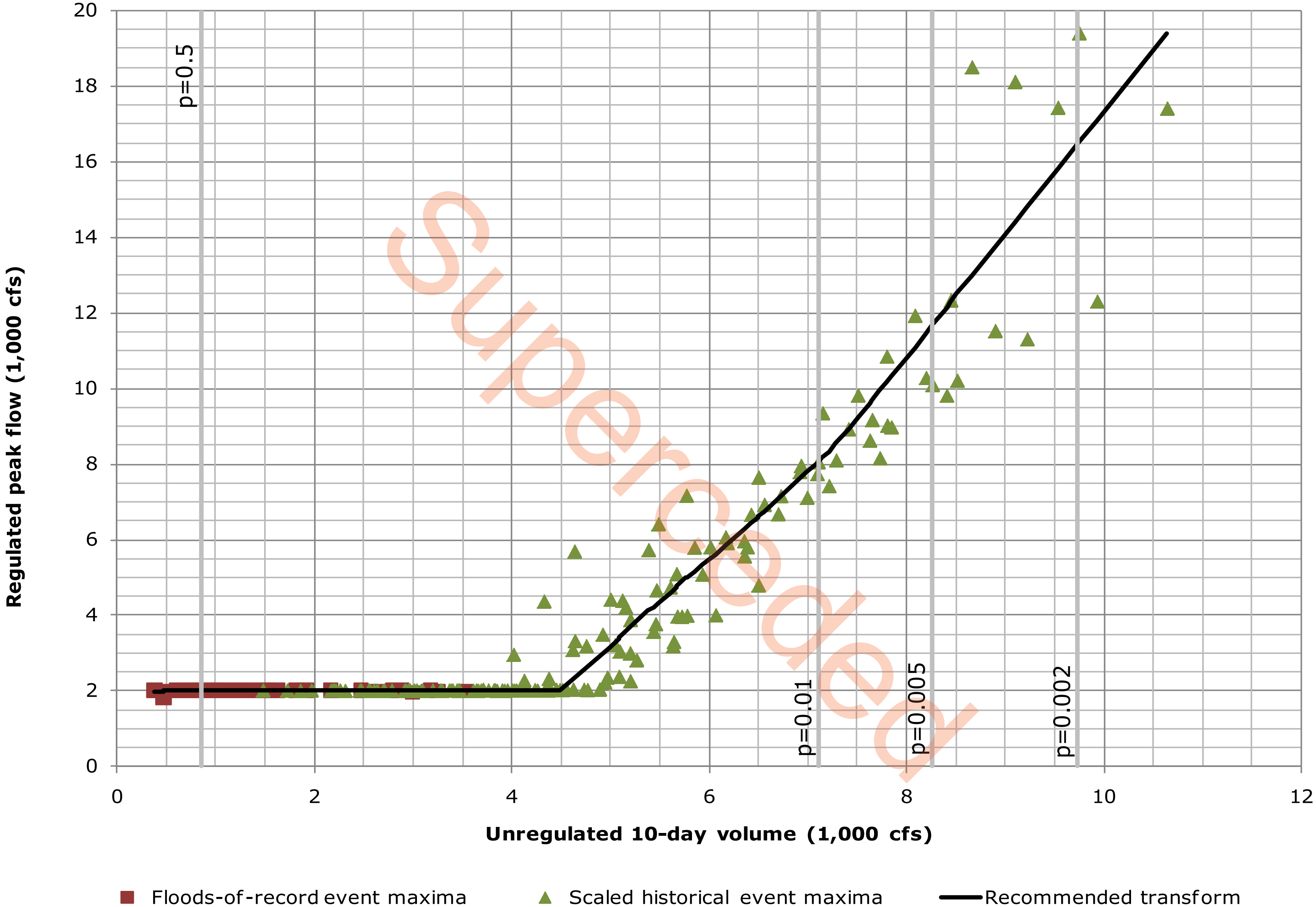

Figure 13. Unregulated-regulated flow transform: Farmington Reservoir

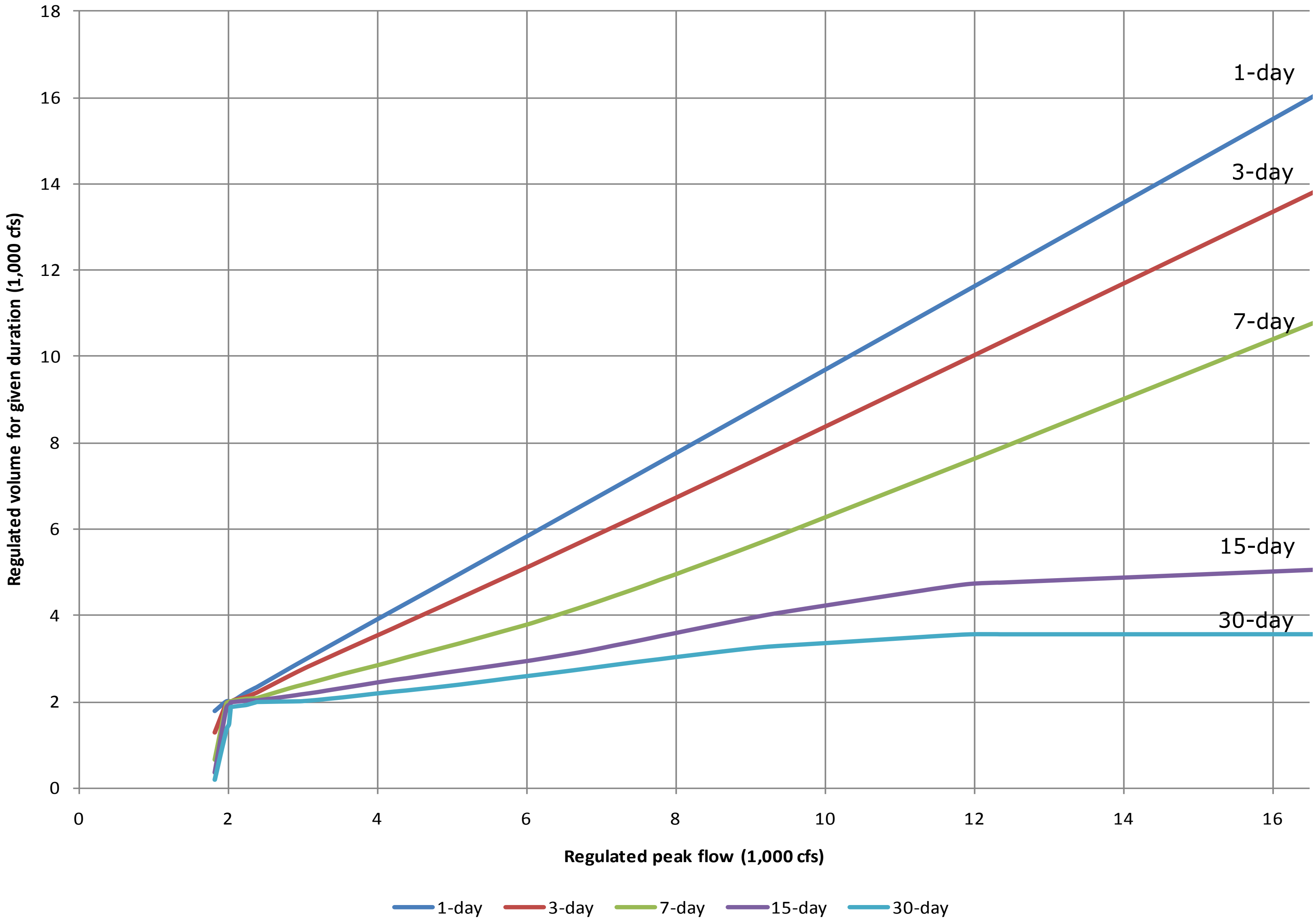

Figure 14. Family of regulated characteristic curves: Farmington Reservoir

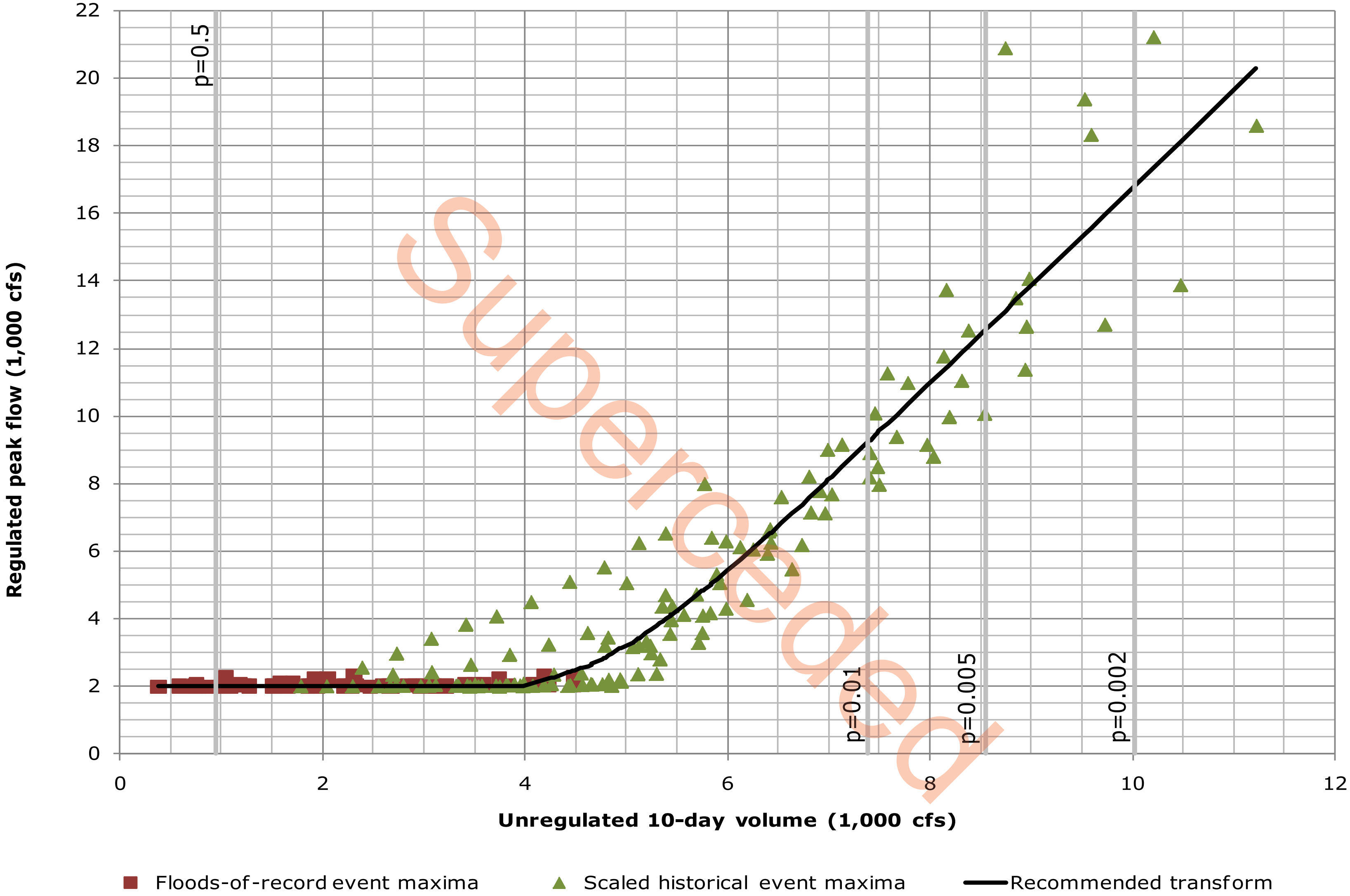

Figure 15. Unregulated-regulated flow transform: Littlejohn Creek at Farmington, CA

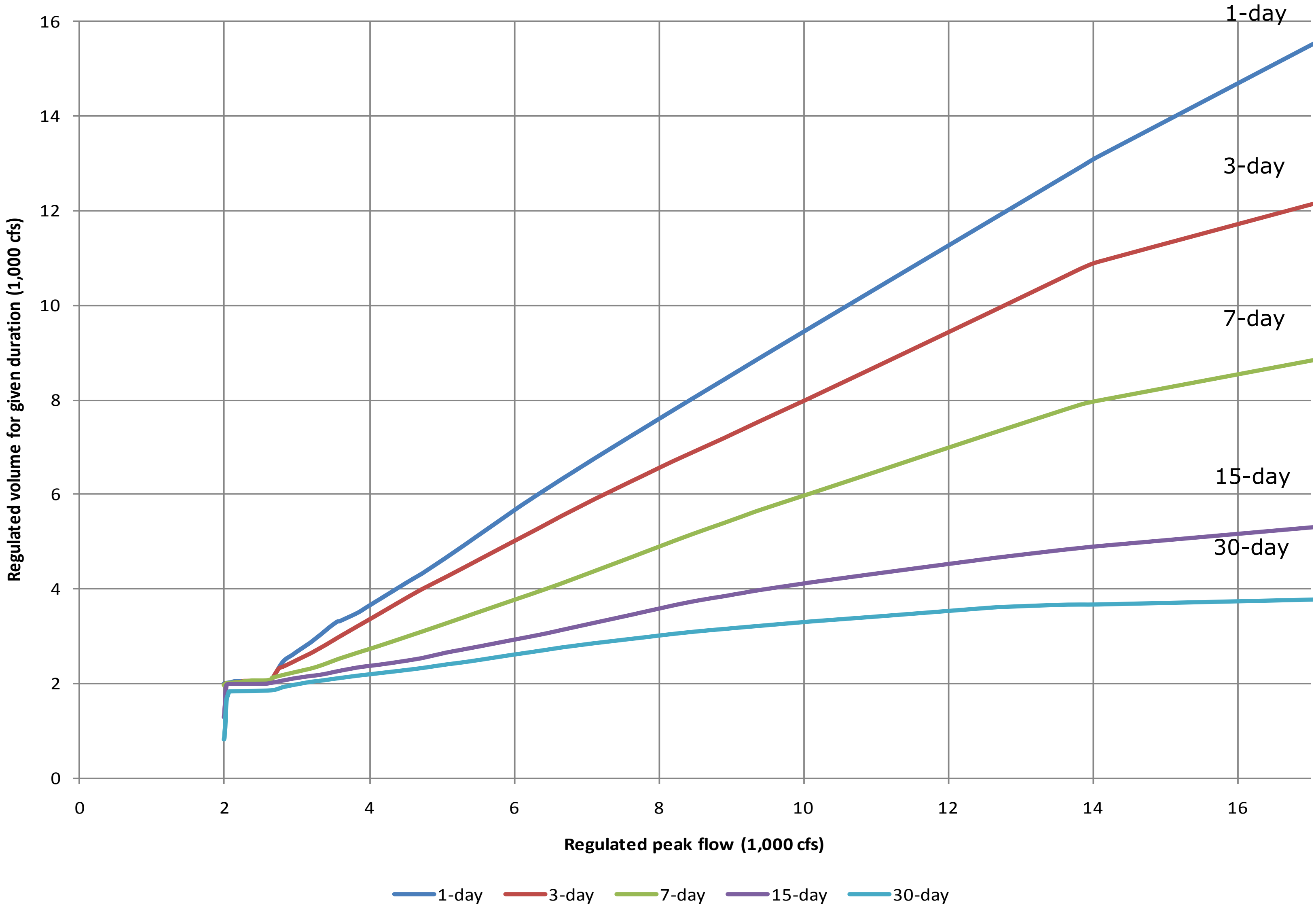
1-day
3-day
7-day
15-day
30-day
0
2
4
6
8
10
12
14
16
Regulated volume for given duration (1,000 cfs)
Regulated peak flow (1,000 cfs)
1-day
3-day
7-day
15-day
30-day

Figure 16. Family of regulated characteristic curves: Littlejohn Creek at Farmington, CA

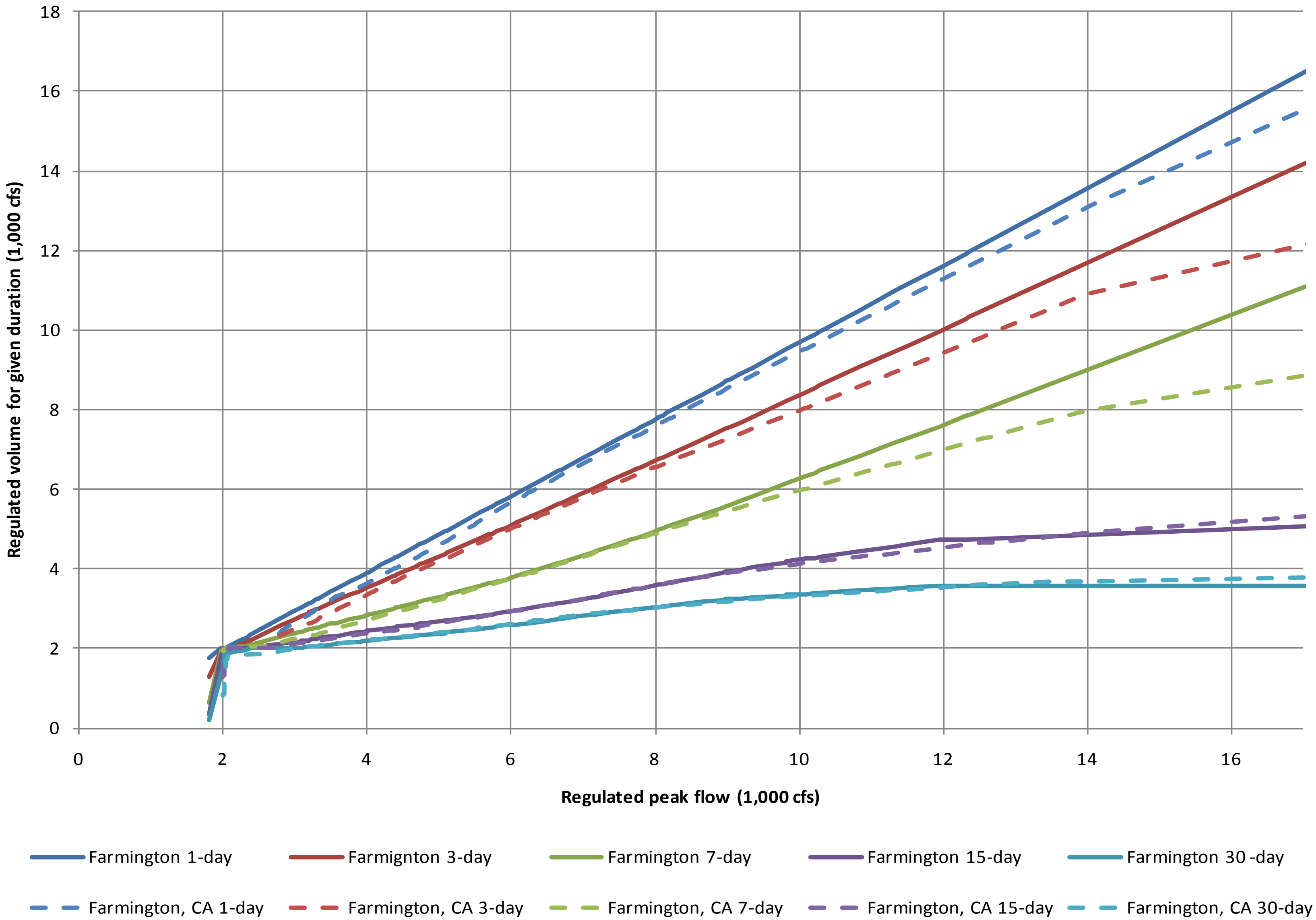

Figure 17. Comparison of the families of characteristic curves for Farmington Reservoir and Farmington, CA

Table 7. Regulated peak flow-frequency quantiles: Farmington Reservoir

Annual exceedence probability (1)	1/annual exceedence probability (2)	Regulated peak flow (cfs) (3)
0.500	2	2,000
0.200	5	2,000
0.100	10	2,000
0.050	20	2,000
0.020	50	5,360
0.010	100	8,077
0.005	200	11,671
0.002	500	16,444

Superceded - table 7

Table 8. Regulated peak flow values and associated volumes: Farmington Reservoir

Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes[1] (as average flow for given duration)				
		1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)	30-day (cfs) (7)
0.500	2,000	2,000	1,994	1,987	1,910	1,491
0.200	2,000	2,000	1,994	1,987	1,910	1,491
0.100	2,000	2,000	1,994	1,987	1,910	1,491
0.050	2,000	2,000	1,994	1,987	1,910	1,491
0.020	5,360	5,213	4,601	3,469	2,776	2,458
0.010	8,077	7,833	6,783	4,996	3,614	3,052
0.005	11,671	11,307	9,746	7,397	4,662	3,536
0.002	16,444	15,928	13,704	10,695	5,043	3,563

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at Farmington Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

Table 9. Regulated peak flow-frequency quantiles: Littlejohn Creek at Farmington, CA

Annual exceedence probability (1)	**1/annual exceedence probability (2)**	**Regulated peak flow (cfs) (3)**
0.500	2	2,000
0.200	5	2,000
0.100	10	2,000
0.050	20	2,633
0.020	50	5,964
0.010	100	9,231
0.005	200	12,548
0.002	500	16,839

Table 10. Regulated peak flow values and associated volumes: Littlejohn Creek at Farmington, CA

Annual exceedence probability of regulated peak flow (1)	**Regulated peak flow (cfs) (2)**	**Associated volumes[1] (as average flow for given duration)**				
		1-day (cfs) (3)	**3-day (cfs) (4)**	**7-day (cfs) (5)**	**15-day (cfs) (6)**	**30-day (cfs) (7)**
0.500	2,000	2,000	1,967	1,960	1,296	827
0.200	2,000	2,000	1,967	1,960	1,296	827
0.100	2,000	2,000	1,967	1,960	1,296	827
0.050	2,633	2,073	2,073	2,073	2,016	1,869
0.020	5,964	5,622	4,978	3,742	2,923	2,616
0.010	9,231	8,741	7,430	5,576	3,943	3,211
0.005	12,548	11,773	9,833	7,268	4,649	3,613
0.002	16,839	15,385	12,070	8,790	5,291	3,781

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at Farmington Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

Expected hydrograph properties

The expected (design) hydrograph for a given exceedence probability is a Farmington Reservoir outflow hydrograph with a peak flow that matched the regulated flow-frequency curve (as shown in Table 7) and with associated volumes matching those from the family of characteristic curves corresponding to the given regulated peak flow (as shown in Table 8). The properties of the expected hydrographs for the p=0.5, p=0.2, p=0.1, p=0.05, p=0.02, p=0.01, p=0.005, and the p=0.002 exceedence probabilities are shown in Table 11.

An expected hydrograph can be formed by applying these properties to a specific hydrograph shape. As part of future work, we will identify specific historical event patterns to which the expected hydrograph properties can be applied. For this identification, we will follow the example event selection procedure provided in the *CVHS Product uses* document (USACE 2009c).

Options for expected hydrograph development and application using study products were submitted by Ford Engineers to the Corps on June 23, 2010. From that memorandum, the Corps selection Option 1: Selected event-based reservoir release hydrographs.

Table 11. Expected hydrograph properties: Farmington Reservoir outflow

Annual exceedence probability of regulated peak flow (1)	1/annual exceedence probability of peak flow (2)	Regulated peak flow (cfs) (3)	Associated volumes[1] (as average flow for given duration)				
			1-day (cfs) (4)	3-day (cfs) (5)	7-day (cfs) (6)	15-day (cfs) (7)	30-day (cfs) (8)
0.500	2	2,000	2,000	1,994	1,987	1,910	1,491
0.200	5	2,000	2,000	1,994	1,987	1,910	1,491
0.100	10	2,000	2,000	1,994	1,987	1,910	1,491
0.050	20	2,000	2,000	1,994	1,987	1,910	1,491
0.020	50	5,360	5,213	4,601	3,469	2,776	2,458
0.010	100	8,077	7,833	6,783	4,996	3,614	3,052
0.005	200	11,671	11,307	9,746	7,397	4,662	3,536
0.002	500	16,444	15,928	13,704	10,695	5,043	3,563

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at Farmington Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

Results

The results of this frequency analysis include:

- Unregulated frequency curves for Farmington Reservoir (as shown in Figure 9).
- Unregulated frequency curves for Littlejohn Creek at Farmington, CA (as shown in Figure 10).
- Unregulated-regulated flow transform for Farmington Reservoir (as shown in Figure 13).
- Regulated flow-frequency curve and associated volumes for Farmington Reservoir (as shown in Table 7 and in Table 8).
- Unregulated-regulated flow transform for Littlejohn Creek at Farmington, CA (as shown in Figure 15).
- Regulated flow-frequency curve and associated volumes for Littlejohn Creek at Farmington, CA (as shown in Table 9 and in Table 10).
- Expected hydrograph properties for Farmington Reservoir (as shown in Table 11).

In addition, these intermediate data are included with the original report on DVD:

- HEC-DSS time series of the floods-of-records.
- HEC-DSS time series of the scaled historical floods.
- HEC-DSS time series of developed local flows below Farmington Reservoir (detailed in Attachment 2).
- The tabulated event maxima datasets for the 2 analysis sites.
- Simulated reservoir releases and routed flows from the HEC-ResSim reservoir simulation model.
- Tabulated unregulated-regulated flow transforms for the 2 analysis sites.
- Tabulated families of regulated characteristic curves for the 2 analysis sites.

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USACE. (Forthcoming 2). *Central Valley hydrology study (CVHS) technical procedures document Attachment B: Unregulated time series development*, in preparation by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (Forthcoming 3). *Central Valley hydrology study (CVHS) Technical procedures document Attachment C: Regulated time series development*, in preparation by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (Forthcoming 4). *Central Valley hydrology study (CVHS) Technical procedures document Attachment D: Flow frequency analysis*, in preparation by David Ford Consulting Engineers, Inc., Sacramento, CA.

USACE. (Forthcoming 5). *Central Valley hydrology study (CVHS) Technical procedures document Attachment E, Development of flow and stage transforms*, in preparation by David Ford Consulting Engineers, Inc., Sacramento, CA.

Attachment 1: Correspondence of procedural steps

Table 12 shows how the procedural steps in this document correspond to the steps in the *Procedures document* and the *Technical procedures document*.

Table 12. Correspondence of procedural steps for the LSJR FS, the CVHS "Procedures document," and the CVHS "Technical procedures document"

This step in the hydrologic analysis at Farmington Reservoir… (1)	**Corresponds to this action in the *Procedures document*… (2)**	**And/or this action in the *Technical procedures document*… (3)**
Develop unregulated flow time series	Task 3.0	Attachment B: Unregulated flow time series development
• Estimate local flows	Task 3.2	• Application and distribution of local flows
• Route and complete unregulated flow time series at analysis locations	Task 3.3	• Procedures for routing flows through the system
Develop unregulated frequency curves	Task 4.0	Attachment D: Frequency analysis
Develop regulated flow time series	Task 5.0	Attachment C: Regulated time series development
• Identify floods-of-record • Scaling of historical reservoir inflows	Task 6.2	Attachment E: Development of flow and stage transforms • Determination of historical event scaling for extrapolating unregulated-regulated flow transform
• Simulation of reservoir releases for historical and scaled events	Task 5.1, Task 6.2	• Procedures for routing regulated flows through the system
Develop flow transforms	Task 6.0	Attachment E: Development of flow and stage transforms
• Identify annual maximum series	Task 6.1	—
• Assess reservoir critical duration	—	Attachment E: Development of flow and stage transforms • Identification of critical duration at analysis points Attachment F: Procedure for critical duration calculation

This step in the hydrologic analysis at Farmington Reservoir… (1)	Corresponds to this action in the *Procedures document*… (2)	And / or this action in the *Technical procedures document*… (3)
• Fit unregulated-regulated flow transform • Fit family of regulated characteristic curves	Task 6.3	Attachment E: Development of flow and stage transforms • Procedure for fitting a "most likely" transform through the datasets
• Apply flow transforms to develop regulated-flow-frequency curves	Task 6.4	—
Develop expected hydrographs[1]	—	—

Notes:

1. Options for expected hydrograph development using study products were submitted by Ford Engineers to the Corps on June 23, 2010. From that memorandum, the Corps selection Option 1: Selected event-based reservoir release hydrographs.

Attachment 2: Littlejohn Creek local flow development

Overview

For Littlejohn Creek, we estimated local flows for the area between Farmington Reservoir and Farmington, CA, shown in Figure 8. For this area, we used 2 options to estimate local flow:

- Option 1. Direct calculation of local flow.
- Option 2: Estimation of local flow as a function of Farmington Reservoir inflow. Note: the Corps currently estimates local flow as 0.04 times reservoir inflow.

Option 1 is the most accurate option for local flow estimation. To determine which of the other 2 options for local flow estimation is more appropriate to use, we:

- Reviewed the streamgage and reservoir inflow data provided by the Corps. In Table 13 we list the streamgages that were used in estimating local flows on Littlejohn Creek. Column 1 lists the streamgage ID whose corresponding name is listed in column 2, column 3 lists the data type (e.g., daily or hourly), column 4 lists the applicable time period of the streamgage data, and column 5 lists notes on the data.
- Coordinated with Corps staff regarding streamgage data quality.
- Identified the data type (e.g., daily or hourly) of the provided data.
- Identified the overlapping time periods for each streamgage by time step.
- Estimated local flow by direct calculation (Option 1).
- Compared the directly calculated local flow time series to Farmington Reservoir inflows.
- Identified, for Option 2, alternative functions for estimating local flow including:
    - Direct multipliers based on ratios of peak flows for selected large events.
    - Direct multipliers based on drainage area ratios.
    - Linear functions determined by regression analysis.
    - Exponential functions determined by regression analysis.
    - Linear functions of logarithmic transforms of flow determined by regression.
- Estimated local flow time series using the possible functions identified.
- Estimated a local flow time series using the reservoir inflow and the 0.04 multiplier used by the Corps.
- Compared the estimated local flow time series to the directly calculated local flow time series.

- Identified the function for each option that most reasonably estimates local flows.

Table 13. Streamgages reviewed for use in estimating local flows on Littlejohn Creek: data were provided by Corps on 6/22/2010 as part of the CVHS.

USGS or CDEC ID (1)	Streamgage name (2)	Data type (3)	Time period (water year) (4)	Notes (5)
—	Farmington Reservoir unregulated inflow	Daily	1951-2009	Values computed by Corps.
FRM	Farmington Dam (reservoir outflow)	Daily	1951-2009	
		Hourly	1995-2009	Data starts January 1, 1995.
FRG	Littlejohn Creek at Farmington, CA	Daily	1948-2008	Streamgage data is influenced by regulation.
		Hourly	1995-2008	Data starts January 1, 1995. Streamgage data is influenced by regulation.
—	Duck Creek Diversion	Daily	1952-2009	Diversion began operation in 1951.
		Hourly	1995-2009	Data starts January 1, 1995.
—	Duck Creek near Farmington	Daily	1979-2009	Data starts January 1, 1979.
		Hourly	1995-2009	Data starts January 1, 1995.
—	Rock Creek at Farmington	Daily	1950-2010	Streamgage data is influenced by regulation.
		Hourly	1995-2010	Data starts January 1, 1995. Streamgage data is influenced by regulation.

Event selection for local flow estimation analysis

As previously noted, local flows developed were used to support the development of an unregulated-regulated flow transform and a family of regulated characteristic curves. A key aspect in the development of these was the scaling of the largest events, i.e., the 15 events previously indentified for Littlejohn Creek.

Thus, the local flows estimated for these large events needed to be reasonable and as accurate as possible. To assess this, we used the local flows calculated directly corresponding to the largest events possible as a basis of comparison. Specifically, we used the 1997, 1998, and 2006 water year events whenever possible.

Local flow estimation Option 1: Calculate local flows directly

The preferred option for estimating local flows was to calculate directly flows using streamgage data. In general, this was completed on Littlejohn Creek using known releases from Farmington Reservoir and the observed flows at Farmington, CA. This was completed only for the time periods when data overlap. On Littlejohn Creek this corresponds to all floods events in the period of record, except for the 1969, 1970, and 1973 water year events.

In the case of daily data, local flows were calculated directly by subtracting the reservoir releases and observed diversion diversions from Duck Creek from the gaged flows. Any resulting negative values were then set to 0. Routing of the daily observed outflows (using the 1-hour hydrologic routing model of Littlejohn Creek) was not necessary because the total travel time between Farmington Reservoir and Farmington, CA, is less than 1-day.

Accepted travel time estimates between Farmington Reservoir and Farmington, CA, are: (1) 3 hours as indicated in the Farmington Reservoir water control manual (Corps 2004), and (2) 1.7 hours as indicated by the sum of Muskingum K value from the HEC-ResSim model provided by the Corps. This shorter travel time was attributed to the availability of hourly streamgage data after 1995 used to calibrate the reservoir simulation and hydrologic routing flood model of Littlejohn Creek, and was adopted for this analysis.

In the case of hourly data, reservoir releases were first routed from Farmington Reservoir downstream to the gage at Farmington, CA. These routed releases and the observed diversions from Duck Creek were then subtracted from the observed flows to calculate local flow directly. Any resulting negative values are then set to 0. We used hydrologic routing to estimate local flows on Littlejohn Creek. Specifically, we used HEC-DSS math utilities and the Muskingum routing parameters from the CVHS HEC-ResSim model as shown in Table 14. In Table 14, column 2 lists the reach, column 3 lists the Muskingum K values in hours, column 4 lists the Muskingum X, and column 5 the number of subreaches.

In Table 15 we summarize how local flows were calculated directly by time period and data type. In Table 15, column 2 lists the data type, column 3 the overlapping time period, and column 4 the components for calculating local flows.

In Figure 18 through Figure 20 we compared the daily and hourly inferred local flows for the 1997, 1998, and 2006 water year events. (These events are the 3 largest of the overlapping time period for which we could calculate both daily and hourly local flows.) In Figure 18 through Figure 20 the daily local flows are shown in red, the hourly local flows in blue, and the daily differences in their volumes (daily local flows minus hourly local flows) in green. From these comparisons we see (1) that the timing of the hourly and daily local flows are similar, and (2) the differences in volume appear to be greatest around the largest local flows associated with the event. These differences in volumes are small compared to the total volume of unregulated inflow to Farmington Reservoir.

Table 14. Littlejohn Creek Muskingum routing parameters between Farmington Reservoir and Farmington, CA

Reach (1)	**Muskingum K (hours) (2)**	**Muskingum X (3)**	**Number of subreaches (4)**
Farmington Reservoir to Farmington, CA	1.7	0.2	1

Table 15. Summary of direct calculation of local flows on Littlejohn Creek

ID (1)	**Data type (2)**	**Overlapping time period[1] (water year) (3)**	**Calculate local flows directly by:[2] (4)**
1	Daily	1951-1968 1971-1972 1974-2008	Subtracting (1) known outflows from Farmington Reservoir and (2) observed flows from Duck Creek, via the Duck Creek diversion, from observed flows on Littlejohn Creek at Farmington, CA
2	Hourly	1996-2008	Routing known outflows from Farmington Reservoir, then subtracting (1) the routed outflows and (2) observed flows from Duck Creek, via the Duck Creek diversion, from observed flows on Littlejohn Creek at Farmington, CA

Notes:

1. Because of missing values, local flow may not be calculated directly for the entire period listed. In such cases flows are either interpolated using the directly calculated flow, or Option 2 or Option 3 depending on data availability.
2. Any resultant negative values were set to 0.

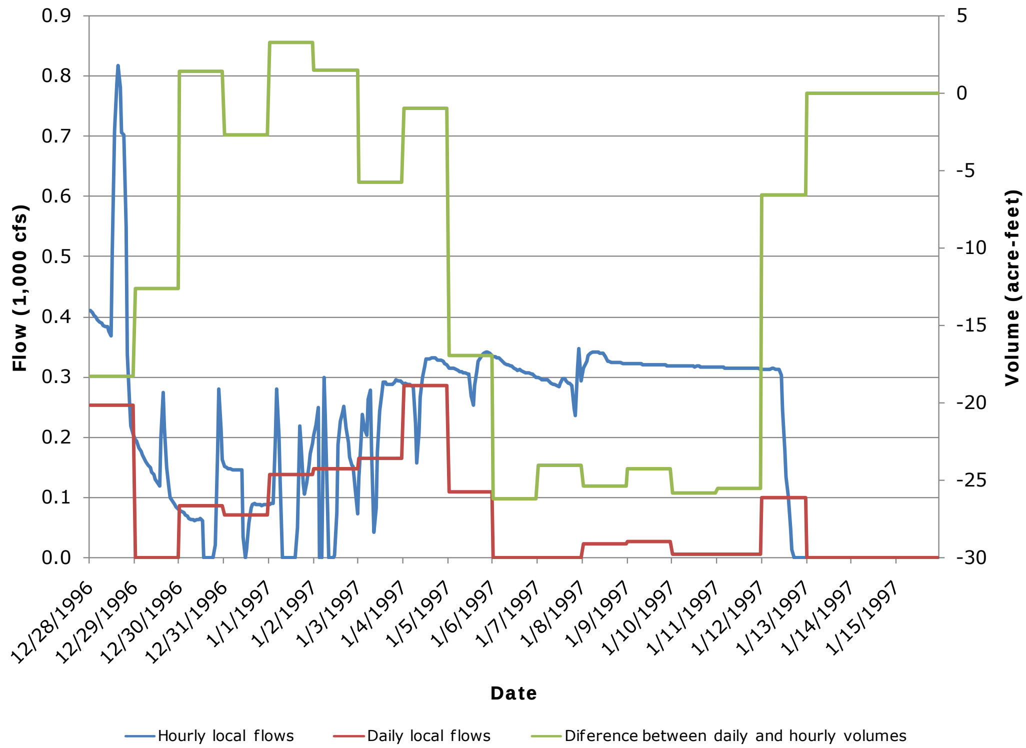


Figure 18. Littlejohn Creek 1997 event directly calculated local flows

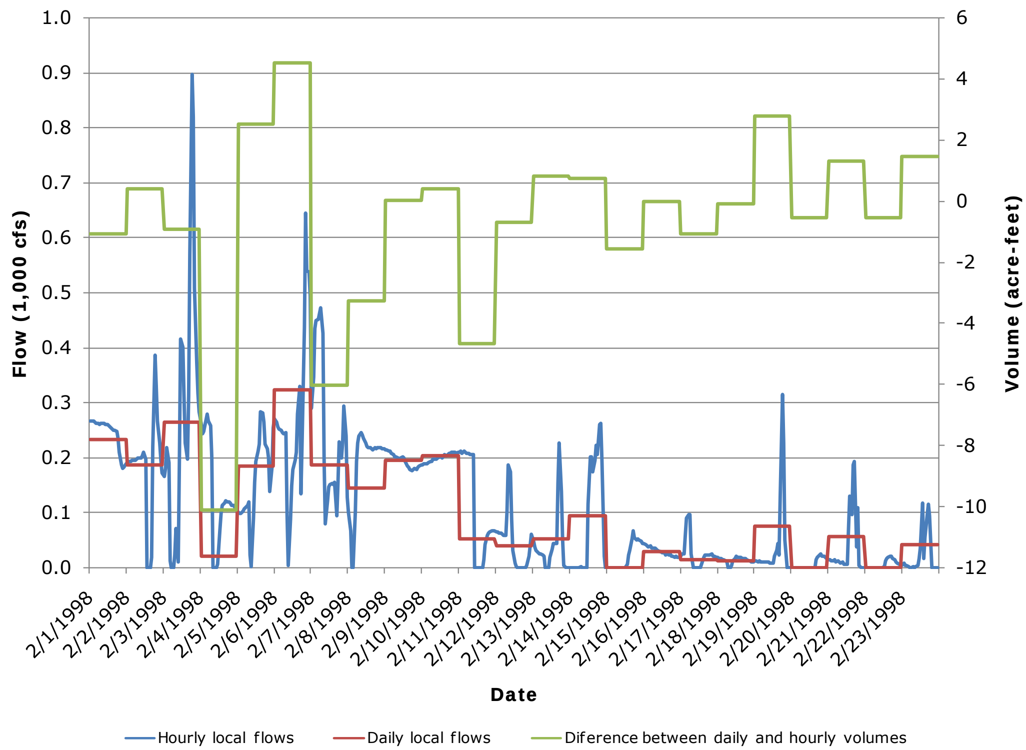


Figure 19. Littlejohn Creek 1998 event directly calculated local flows

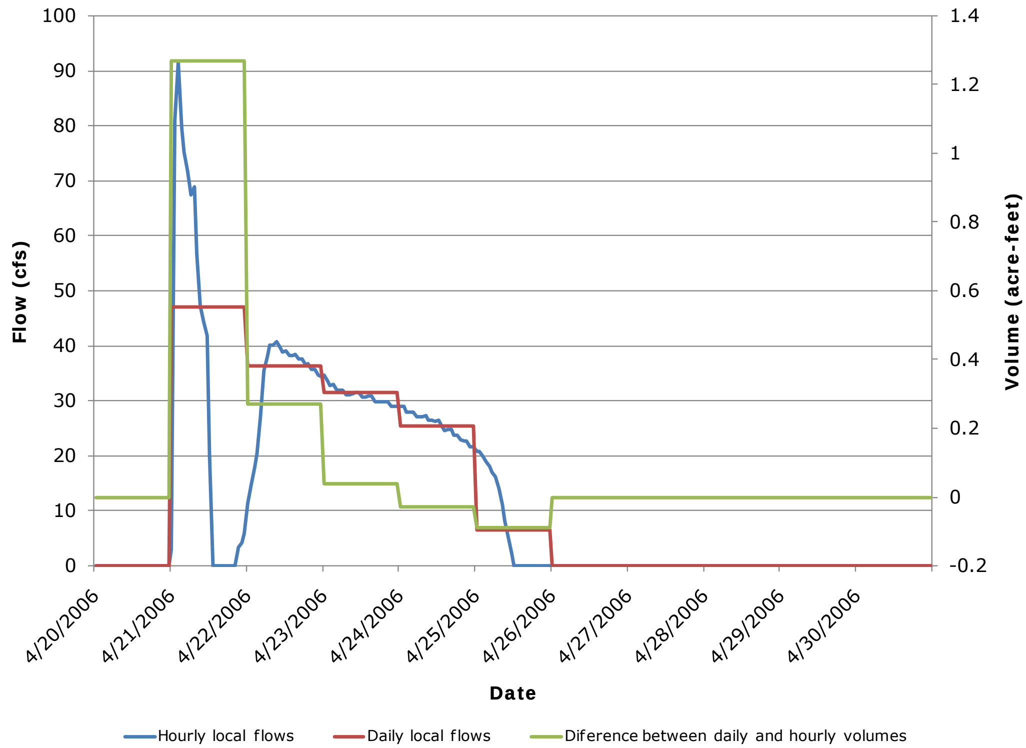


Figure 20. Littlejohn Creek 2006 event directly calculated local flows

Local flow estimation Option 2: Estimate local flows as a function of unregulated inflow to Farmington Reservoir

In the cases where local flows could not be calculated directly, we estimated local flows using reservoir inflows. As noted above, the Corps already estimates local flows using coefficients for reservoir operations on Littlejohn Creek as 0.04 times the reservoir inflow. Because the estimation of local flows is important to simulate accurately reservoir operations we need to either (1) verify the coefficients used by the Corps to estimate such flows, or (2) adopt new coefficients. We completed this task by:

- Calculating local flows directly as detailed in the previous subsection.
- Comparing the directly calculated local flow time series to observed flows on reservoir inflows for selected large events occurring in the overlapping period of record.
- Identifying an average ratio of maximum 1-day inflows to directly calculated peak local flows for selected large events.
- Estimating local flow time series using the average ratio identified as a multiplier of unregulated reservoir inflow.
- Estimating local flow time series using a drainage area ratio between the local flow area and watershed above the reservoir as a multiplier to reservoir inflows.
- Completing regression analyses that relate the directly calculated local flows to the reservoir inflow for the overlapping periods of record.

- Identifying the best fitted functions from the regression analysis for estimation of local flows.
- Estimating local flow time series using the identified functions.
- Estimating a local flow time series using the unregulated reservoir inflow and the 0.04 multiplier used by the Corps.
- Comparing the estimated local flow time series to the directly calculated local flow time series.
- Identifying the function that most reasonably estimates local flows.

Based on this analysis, we identified the best relation for estimating local flows using reservoir inflow to be the function currently used by the Corps. Thus, we estimated local flows as:

$$Q_{Local} = 0.04\left(Q_{FRM}\right) \tag{2}$$

where Q_{Local} is the local flow estimate for a given time, and Q_{FRM} is the unregulated inflow to Farmington Reservoir. The Corps estimates local flows for the purpose of real-time reservoir operations using this option (John High, personal communication, 11/9/2009) and this is the option used to estimate local flows in the Comp Study (USACE 2002).

All estimated local flows using this option were on a daily basis. We did not lag or route the estimated flows because: (1) synthesizing a shorter time step is not required for frequency analysis, and (2) the travel time between the Farmington Reservoir and Farmington, CA, is approximately 2 hours, which is less than the 1-day time step of the reservoir inflows.

Local flow estimation details

The selected estimation approaches, in order of best estimate of local flow, are:

- Option 1. Calculate local flow directly using known releases from Farmington Reservoir and the observed flows at Farmington, CA, routing hourly flows as necessary. Note in the case of missing streamgage data, local flows values were interpolated as needed.
- Option 2. Estimate local flow as 0.04 times the unregulated inflow to Farmington Reservoir.

We detail the development of the local flow time series for Farmington Reservoir in Table 16. Column 1 notes the time period for which the option listed in column 3 will be used to estimate local flow, and column 2 lists the time step (hourly or daily) of the developed local flow time series. We interpolated local flows using other estimated local flows as appropriate. The hourly and daily time series were combined and these finalized time series stored as hourly data in HEC-DSS.

Table 16. Local flow time series calculation details by time period

Period (date) (1)	**Time step (2)**	**Approach to be used[1] (3)**
10/1/1950—9/30/1951	Daily	Option 1: directly calculate local flow. Note that the Duck Creek diversion was not in operation during this time period.
10/1/1951-1/6/1969	Daily	Option 1: directly calculate local flow
1/7/1969-3/29/1969	Daily	Option 2: 0.04 times reservoir inflow.
3/30/1969-1/10/1970	Daily	Option 1: directly calculate local flow
1/11/1970-3/31/1970	Daily	Option 2: 0.04 times reservoir inflow.
4/1/1970-1/7/1973	Daily	Option 1: directly calculate local flow.
1/8/1973-4/5/1973	Daily	Option 2: 0.04 times reservoir inflow.
4/6/1973-5/3/1978	Daily	Option 1: directly calculate local flow.
5/4/1978-9/30/1978	Daily	Assume 0 local flow.
10/1/1978-10/31/1978	Daily	Option 1: directly calculate local flow.
11/1/1978-1/10/1979	Daily	Assume 0 local flow.
1/11/1979-4/5/1979	Daily	Option 1: directly calculate local flow.
4/6/1979-9/24/1979	Daily	Assume 0 local flow.
9/25/1979-9/30/1991	Daily	Option 1: directly calculate local flow.
10/1/1991-12/31/1991	Daily	Assume 0 local flow.
1/1/1992-12/31/1994	Daily	Option 1: directly calculate local flow.
1/1/1995-9/27/1995	Hourly	Option 1: directly calculate local flow.
9/28/1995-12/18/1995	Daily	Option 1: directly calculate local flow.
12/19/1995-12/28/2008	Hourly	Option 1: directly calculate local flow.

Attachment 3: Annual maximum series for unregulated frequency curves

Here we list the series of annual maximum unregulated volume values that we used in development of the unregulated frequency curves for Farmington Reservoir and at Farmington, CA. In addition, we include here the unregulated peak inflow annual maximum series for Farmington Reservoir. Development of a peak flow-frequency curve is not required for development of the regulated flow-frequency curves. However, we developed such curves for completeness.

Annual maximum series

For the Farmington Reservoir, the unregulated reservoir inflow time series was used as the basis of the unregulated frequency analysis. The Corps provided the finalized unregulated inflow time series for Farmington Reservoir on 7/12/2010. From this time series, we extracted the 1-, 3-, 7-, 15-, and 30-day volume data. We list these values for Farmington Reservoir in Table 17. In the table, column 1 lists the water year, and columns 2 through 11 list the date, if available, and the volume, as average flow for the given duration, in cfs. The dates listed in Table 17 correspond to the start of the duration.

To develop annual maximum series for Farmington Reservoir's operation point on Littlejohn Creek at Farmington, CA, we combined the unregulated inflow time series with the estimated local flows by adding the 2 time series together using HEC-DSS math utilities. Note that we did not route the unregulated reservoir inflows because the travel time between the reservoir and the operation point is less than the time step of the inflows: 1 day.

Using these data, we computed the 1-, 3-, 7-, 15-, and 30-day volume-duration data using HEC-SSP version 1.1. We list these values for Farmington, CA, in Table 18. In the table, column 1 lists the water year, and columns 2 through 11 list the date, if available, and the volume, as average flow for the given duration, in cfs. The dates listed in Table 18 correspond to the start of the duration.

In addition, we reviewed the computed values for consistency. Specifically, we checked that the extracted value for a given duration is less than the values associated with each shorter duration in a given water year. For both analysis locations, we found that the computed values for each water year decrease as duration increases.

Table 17. Farmington Reservoir annual maximum series for unregulated volume-frequency analysis

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1951	12/8/1950	5,284	12/9/1950	4,045	12/10/1950	2,762	12/17/1950	1,605	12/17/1950	1,057
1952	3/15/1952	5,019	1/27/1952	3,351	1/29/1952	2,219	1/28/1952	1,418	1/28/1952	1,013
1953	1/14/1953	725	1/15/1953	450	1/20/1953	398	1/21/1953	316	1/28/1953	210
1954	3/17/1954	723	3/19/1954	417	3/23/1954	290	3/31/1954	166	4/14/1954	97
1955	1/1/1955	3,556	1/18/1955	1,945	1/21/1955	1,245	1/24/1955	701	1/30/1955	530
1956	12/24/1955	8,497	12/25/1955	7,413	12/28/1955	3,765	1/6/1956	2,100	1/21/1956	1,582
1957	3/5/1957	2,232	3/7/1957	1,086	3/11/1957	523	3/17/1957	263	3/30/1957	135
1958	4/3/1958	7,272	4/3/1958	6,913	4/6/1958	3,945	4/4/1958	2,234	4/12/1958	1,470
1959	2/16/1959	1,419	2/18/1959	1,218	2/22/1959	851	2/25/1959	541	3/12/1959	307
1960	2/10/1960	1,402	2/12/1960	665	2/13/1960	459	2/21/1960	286	3/2/1960	157
1961	2/2/1961	102	2/4/1961	78	2/8/1961	61	2/15/1961	38	2/15/1961	19
1962	2/15/1962	5,086	2/15/1962	2,914	2/16/1962	2,439	2/23/1962	1,370	3/10/1962	911
1963	2/13/1963	3,205	2/13/1963	1,518	2/16/1963	1,028	2/15/1963	729	4/26/1963	467
1964	1/22/1964	898	1/24/1964	749	1/27/1964	486	1/26/1964	327	2/10/1964	172
1965	12/26/1964	8,760	12/26/1964	6,357	12/28/1964	4,162	1/6/1965	2,462	1/20/1965	1,447
1966	1/30/1966	2,071	12/31/1965	1,246	1/4/1966	643	2/13/1966	438	2/27/1966	252
1967	1/22/1967	4,324	4/20/1967	2,392	4/24/1967	1,956	4/21/1967	1,368	4/29/1967	948
1968	2/21/1968	1,241	2/22/1968	699	2/23/1968	424	3/2/1968	240	3/17/1968	162
1969	1/21/1969	3,707	1/23/1969	3,459	1/27/1969	2,898	1/27/1969	2,383	2/11/1969	1,565
1970	1/21/1970	3,953	1/22/1970	3,689	1/25/1970	3,284	1/28/1970	2,577	2/6/1970	1,399
1971	11/29/1970	2,624	12/1/1970	1,482	12/5/1970	1,133	12/12/1970	590	12/28/1970	408
1972	12/25/1971	1,267	12/27/1971	891	12/31/1971	649	1/8/1972	328	1/22/1972	170
1973	2/11/1973	5,368	1/16/1973	3,565	1/18/1973	2,260	1/23/1973	1,361	2/11/1973	961
1974	3/2/1974	4,749	4/4/1974	1,931	4/7/1974	1,673	4/13/1974	1,220	4/14/1974	621

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1975	3/22/1975	2,742	2/14/1975	1,273	2/14/1975	911	3/27/1975	704	4/5/1975	495
1976	9/11/1976	10	8/25/1976	5	9/11/1976	2	9/6/1976	2	9/17/1976	2
1977	10/1/1976	-	10/1/1976	-	10/1/1976	-	10/1/1976	-	10/1/1976	-
1978	2/9/1978	3,447	2/9/1978	2,760	2/13/1978	1,534	2/14/1978	850	3/7/1978	788
1979	2/21/1979	5,080	2/23/1979	3,581	2/24/1979	2,450	3/4/1979	1,589	3/14/1979	923
1980	1/12/1980	4,921	1/14/1980	3,899	1/17/1980	2,449	1/25/1980	1,289	2/8/1980	667
1981	1/29/1981	3,890	1/30/1981	1,783	2/2/1981	933	3/30/1981	496	4/4/1981	325
1982	3/31/1982	6,522	2/17/1982	4,434	1/6/1982	2,498	4/12/1982	1,499	4/12/1982	1,202
1983	11/30/1982	6,620	1/24/1983	4,727	1/28/1983	3,243	2/1/1983	2,093	2/15/1983	1,539
1984	12/25/1983	5,755	12/26/1983	3,764	12/28/1983	1,883	1/1/1984	941	1/7/1984	554
1985	2/8/1985	2,411	2/10/1985	1,367	2/10/1985	639	2/10/1985	345	12/20/1984	237
1986	2/19/1986	9,555	2/19/1986	7,662	2/20/1986	4,420	2/24/1986	2,195	3/16/1986	1,522
1987	3/6/1987	2,891	3/7/1987	1,389	3/11/1987	643	3/19/1987	345	4/3/1987	202
1988	1/18/1988	63	1/20/1988	34	1/23/1988	16	1/23/1988	8	1/23/1988	4
1989	3/4/1989	45	3/5/1989	35	3/9/1989	16	3/16/1989	13	4/1/1989	9
1990	4/16/1990	25	4/18/1990	25	4/21/1990	25	4/29/1990	24	3/22/1990	19
1991	3/26/1991	2,718	3/26/1991	2,013	3/30/1991	1,264	4/1/1991	820	4/11/1991	434
1992	2/15/1992	4,517	2/15/1992	2,115	2/17/1992	1,363	2/25/1992	681	3/11/1992	410
1993	1/13/1993	2,697	1/15/1993	1,797	1/18/1993	1,528	1/22/1993	1,236	2/10/1993	721
1994	2/20/1994	281	2/22/1994	162	2/25/1994	104	3/4/1994	60	3/10/1994	37
1995	1/27/1995	4,854	3/12/1995	3,641	1/29/1995	2,128	3/24/1995	1,602	2/2/1995	906
1996	2/21/1996	3,941	2/22/1996	3,054	2/25/1996	1,599	3/2/1996	792	2/23/1996	765
1997	1/2/1997	7,777	1/3/1997	4,344	1/27/1997	2,448	1/4/1997	1,598	1/28/1997	1,127
1998	2/3/1998	11,270	2/4/1998	5,253	2/8/1998	4,628	2/16/1998	2,861	2/10/1998	1,831
1999	2/9/1999	4,517	2/10/1999	2,677	2/13/1999	1,423	2/21/1999	891	2/22/1999	519

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
2000	1/25/2000	5,137	2/14/2000	3,934	2/18/2000	2,049	2/26/2000	1,309	3/11/2000	940
2001	3/5/2001	1,390	3/6/2001	770	3/7/2001	376	3/7/2001	258	3/7/2001	167
2002	1/3/2002	2,653	1/4/2002	1,679	1/4/2002	1,355	1/11/2002	657	1/27/2002	390
2003	1/19/2003	254	3/26/2003	200	3/29/2003	177	3/29/2003	105	3/31/2003	70
2004	2/26/2004	1,170	2/28/2004	834	3/3/2004	567	3/3/2004	305	11/26/2003	182
2005	3/23/2005	4,597	3/24/2005	2,436	1/13/2005	1,539	1/13/2005	1,062	1/29/2005	694
2006	4/4/2006	9,912	4/5/2006	6,096	4/6/2006	3,353	1/3/2006	2,048	4/6/2006	1,273
2007	2/27/2007	869	2/28/2007	670	3/1/2007	504	2/28/2007	411	3/7/2007	266
2008	2/3/2008	3,314	1/29/2008	1,949	1/29/2008	1,346	2/5/2008	957	2/20/2008	584

Table 18. Littlejohn Creek at Farmington, CA, annual maximum series for unregulated volume-frequency analysis

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1951	12/8/1950	5,333	12/9/1950	4,091	12/10/1950	2,828	12/17/1950	1,636	12/17/1950	1,076
1952	3/15/1952	5,019	1/27/1952	3,375	1/29/1952	2,234	1/28/1952	1,435	1/28/1952	1,024
1953	1/14/1953	725	1/15/1953	457	1/20/1953	403	1/21/1953	320	1/28/1953	215
1954	3/17/1954	723	3/19/1954	429	3/23/1954	301	3/31/1954	173	4/14/1954	100
1955	1/1/1955	3,556	1/18/1955	2,034	1/21/1955	1,286	1/24/1955	741	1/30/1955	558
1956	12/24/1955	9,011	12/25/1955	7,994	12/28/1955	4,097	1/6/1956	2,284	1/21/1956	1,697
1957	3/5/1957	2,232	3/7/1957	1,086	3/11/1957	523	3/17/1957	263	3/30/1957	136
1958	4/3/1958	7,553	4/3/1958	7,006	4/6/1958	3,985	4/4/1958	2,281	4/12/1958	1,501
1959	2/17/1959	1,652	2/18/1959	1,388	2/22/1959	1,020	2/25/1959	663	3/12/1959	368
1960	2/10/1960	1,402	2/12/1960	706	2/13/1960	496	2/21/1960	303	3/6/1960	166
1961	2/2/1961	102	2/4/1961	78	2/8/1961	61	2/15/1961	38	2/15/1961	19
1962	2/15/1962	5,097	2/15/1962	2,973	2/16/1962	2,464	2/23/1962	1,386	3/10/1962	932
1963	2/13/1963	3,205	4/16/1963	1,626	2/16/1963	1,036	2/15/1963	752	4/26/1963	541
1964	1/23/1964	1,624	1/24/1964	1,308	1/27/1964	788	1/26/1964	463	2/10/1964	254
1965	12/26/1964	8,760	12/26/1964	6,362	12/28/1964	4,182	1/6/1965	2,476	1/20/1965	1,456
1966	1/30/1966	2,110	12/31/1965	1,246	1/4/1966	656	2/13/1966	469	2/27/1966	267
1967	1/22/1967	4,324	4/20/1967	2,392	4/24/1967	1,999	4/21/1967	1,406	4/29/1967	978
1968	2/21/1968	1,241	2/22/1968	699	2/23/1968	424	3/2/1968	240	3/17/1968	162
1969	1/22/1969	5,299	1/23/1969	5,221	1/27/1969	4,543	1/29/1969	3,713	2/11/1969	2,617
1970	1/23/1970	5,075	1/23/1970	4,886	1/24/1970	4,578	1/28/1970	3,612	2/12/1970	1,968
1971	11/29/1970	2,624	12/1/1970	1,482	12/5/1970	1,149	12/12/1970	641	12/28/1970	448
1972	12/25/1971	1,267	12/27/1971	900	12/31/1971	661	1/8/1972	334	1/22/1972	173
1973	2/11/1973	6,244	1/16/1973	4,240	2/17/1973	3,445	2/20/1973	2,298	3/10/1973	1,639
1974	3/2/1974	4,749	4/4/1974	1,931	4/7/1974	1,679	4/13/1974	1,223	4/14/1974	628

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1975	3/22/1975	2,742	2/14/1975	1,278	2/14/1975	916	3/27/1975	721	4/5/1975	515
1976	9/11/1976	43	9/13/1976	27	10/5/1975	25	10/13/1975	25	10/28/1975	25
1977	8/8/1977	75	8/9/1977	63	9/4/1977	48	9/6/1977	46	9/5/1977	40
1978	2/9/1978	3,517	2/9/1978	2,829	2/13/1978	1,586	2/15/1978	883	3/7/1978	807
1979	2/21/1979	5,163	2/22/1979	3,664	2/24/1979	2,493	3/4/1979	1,609	3/14/1979	933
1980	1/12/1980	4,980	1/14/1980	3,967	1/17/1980	2,486	1/24/1980	1,307	2/8/1980	705
1981	1/29/1981	3,985	1/30/1981	1,871	2/2/1981	995	2/10/1981	533	4/3/1981	354
1982	3/31/1982	6,522	2/17/1982	4,461	1/6/1982	2,610	4/12/1982	1,532	4/12/1982	1,225
1983	11/30/1982	6,876	1/24/1983	4,813	1/28/1983	3,299	2/1/1983	2,137	2/15/1983	1,565
1984	12/25/1983	5,755	12/26/1983	3,894	12/29/1983	2,036	1/6/1984	1,083	1/8/1984	688
1985	2/8/1985	2,419	2/10/1985	1,479	2/14/1985	751	2/22/1985	441	12/23/1984	287
1986	2/17/1986	9,786	2/19/1986	7,897	2/20/1986	4,612	2/21/1986	2,343	3/16/1986	1,634
1987	3/6/1987	3,228	3/7/1987	1,841	3/11/1987	975	3/19/1987	589	4/3/1987	395
1988	1/20/1988	204	1/22/1988	183	1/25/1988	148	2/2/1988	109	8/26/1988	102
1989	3/4/1989	123	3/6/1989	91	3/10/1989	81	3/18/1989	78	4/2/1989	75
1990	3/5/1990	164	2/20/1990	109	2/24/1990	87	3/5/1990	77	3/19/1990	52
1991	3/26/1991	2,718	3/26/1991	2,013	3/30/1991	1,264	4/1/1991	820	4/11/1991	434
1992	2/15/1992	4,517	2/15/1992	2,115	2/17/1992	1,363	2/26/1992	701	3/11/1992	449
1993	1/13/1993	2,810	1/15/1993	1,964	1/18/1993	1,721	1/22/1993	1,419	2/5/1993	884
1994	2/21/1994	429	2/22/1994	414	2/26/1994	360	3/6/1994	320	10/6/1993	234
1995	1/27/1995	4,999	3/12/1995	3,683	1/29/1995	2,308	3/24/1995	1,612	2/3/1995	1,121
1996	2/21/1996	3,977	2/22/1996	3,130	2/25/1996	1,645	3/4/1996	1,001	2/23/1996	880
1997	1/2/1997	7,942	1/3/1997	4,510	1/27/1997	2,453	1/4/1997	1,788	1/28/1997	1,251
1998	2/3/1998	11,547	2/4/1998	5,455	2/8/1998	4,838	2/16/1998	3,008	2/10/1998	2,013
1999	2/9/1999	4,668	2/10/1999	2,736	2/13/1999	1,449	2/21/1999	946	3/8/1999	572

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
2000	1/25/2000	5,149	2/14/2000	3,949	2/18/2000	2,116	2/26/2000	1,366	3/11/2000	976
2001	3/5/2001	1,452	3/6/2001	833	3/10/2001	450	3/9/2001	382	3/10/2001	251
2002	1/3/2002	2,692	1/3/2002	1,752	1/4/2002	1,414	1/11/2002	737	1/26/2002	438
2003	1/1/2003	306	1/3/2003	254	3/29/2003	177	3/29/2003	117	1/21/2003	82
2004	2/26/2004	1,170	2/28/2004	834	3/3/2004	567	3/3/2004	333	3/8/2004	188
2005	3/23/2005	4,597	3/24/2005	2,436	1/13/2005	1,539	1/13/2005	1,062	1/29/2005	694
2006	4/4/2006	9,912	4/5/2006	6,096	4/6/2006	3,353	1/3/2006	2,048	4/6/2006	1,273
2007	2/27/2007	869	2/28/2007	670	3/1/2007	504	2/28/2007	411	3/7/2007	266
2008	2/3/2008	3,345	1/29/2008	1,952	1/29/2008	1,367	2/6/2008	1,004	2/20/2008	608

Peak annual maximum series

To develop the peak inflow annual maximum series for Farmington Reservoir, we reviewed the data provided by the Corps and other sources that contain annual maximum series, including:

- Littlejohn Creek stream group hydrology report (USACE 1983).
- Farmington Reservoir water control manual (USACE 2004), hereafter referred to as Farmington WCM.
- Peak flow data provided by the Corps on 6/11/2010.

We summarize in Table 19 the data we identified for use in developing flow-frequency curves for New Hogan. Column 1 lists the time period for which data were identified, and column 2 lists the source of these data.

Table 19. Data sources of peak inflow annual maximum series data identified for use in developing flow-frequency curves for Farmington Reservoir

Time period (water year) (1)	Data source used (2)
1903-1951[1]	Littlejohn Creek stream group hydrology report (USACE 1983)
1952-2004	Farmington WCM (USACE 2004)

Notes:

1. Intermittent historical data only. Historical information was not used to fit the unregulated inflow frequency curves consistent with current practice for peak flows at this location.

We list the peak inflow values and, where possible, their associated dates of occurrence, for Farmington Reservoir in Table 20. In the table, column 1 lists the water year; column 2 lists the date, if available; and column 3 lists the value in cfs.

We did not develop a peak flow-frequency curve for Littlejohn Creek at Farmington, CA, because a series of annual maximum peak flows at this location is not available. A peak unregulated flow-frequency curve is not required for this analysis.

Table 20. Farmington Reservoir annual maximum peak inflows

Water year (1)	Date of peak inflow (2)	Peak inflow (cfs) (3)
1952	March 1952	11,500
1953-1954	—	—
1955	—	5,700
1956	December 1955	20,000
1957	—	2,400
1958	April 1958	28,900
1959	—	2,390
1960	—	1,100
1961	—	—
1962	—	7,700
1963	—	—
1964	—	2,480
1965	December 1964	18,100
1966	—	—
1967	1/22/1967	8,110
1968	—	—
1969	1/21/1969	7,390
1970-1972	—	—
1973	2/12/1973	7,300
1974	3/2/1974	10,500
1975	3/22/1975	4,400
1976-1979	—	—
1980	1/14/1980	7,900
1981	—	—
1982	3/31/1982	14,411
1983	1/22/1983	16,500
1984	12/25/1983	9,900
1985	—	—
1986	2/19/1986	23,571
1987	3/5/1987	6,779
1988	—	—
1989	3/3/1989	71
1990	—	—
1991	3/24/1991	12,714
1992	2/15/1992	9,595
1993	—	6,823
1994	2/20/1994	807
1995	3/10/1995	12,281

Water year (1)	Date of peak inflow (2)	Peak inflow (cfs) (3)
1996	2/4/1996	10,185
1997	1/2/1997	12,929
1998	2/6/1998	24,830
1999	2/9/1999	8,302
2000	2/12/2000	10,013
2001	2/24/2001	2,465
2002	1/2/2002	6,331
2003	12/16/2002	1,550
2004	2/26/2004	1,992

Attachment 4: Fitting the unregulated frequency curves

Overview

The purpose of this attachment is to describe the steps taken to fit unregulated frequency curves to annual maximum series. We developed unregulated frequency curves following the procedures specified in *Bulletin 17B* (IACWD 1982), guidance detailed in EM 1110-2-1415 (USACE 1993), and the current standards of practice. Specially, we:

- Identified the annual maximum series.
- (Task 4.1) Calculated regional skew values for each duration of interest using relationships developed by the USGS.
- (Task 4.2) Fitted LPIII distributions to the annual maximum series following *Bulletin 17B* procedures and Corps guidance using PeakfqSA, the USGS's flow-frequency software with the expected moments algorithm (EMA) option enabled developed by Tim Cohn of the USGS (Cohn 2007).
- Reviewed and adopted the curves, checking them for consistency and comparing them to previously accepted values.

Regional skew values

Bulletin 17B recommends the use of a regional skew value in fitting LPIII distributions to maintain consistency of frequency curves. *Bulletin 17B* also states that such a value can be developed using regression techniques. For the CVHS, the USGS, in cooperation with the Corps, has developed regression equations for regional skew values (USGS 2010). In general, there are 2 equation forms, 1 for peak flows, and 1 for volumes. The coefficients for the volumes change with duration.

The regional skew associated with peak flows is calculated as:

$$\gamma = -0.62 + 1.30\left(1 - e^{\left(-\left(\frac{Elev}{6500}\right)^{2}\right)}\right) \tag{3}$$

where γ is the regional skew value *Elev* is the average basin elevation in ft (NAVD 88). The associated average variance of prediction (AVP) is 0.14. AVP is analogous to mean square error (MSE) for the purpose of weighting regional and station skew values.

The regional skew associated with volumes is calculated as

$$\gamma = \beta_0 + \beta_1\left(1 - e^{\left(-\left(\frac{Elev}{3600}\right)^{12}\right)}\right) \tag{4}$$

where γ is the regional skew value, *Elev* is the average basin elevation in ft (NAVD 88), and β_0 and β_1 are coefficients based on the duration of interest as shown in Table 21. The associated AVP also varies with duration and is also shown in Table 21.

For this analysis, we used these equations to develop regional skew values for Littlejohn Creek as shown in Table 22. We used GIS tools to compute average basin elevations for use in the regional skew computations.

Table 21. Duration skew equation parameters

Parameter (1)	1-day regional skew (2)	3-day regional skew (3)	7-day regional skew (4)	15-day regional skew (5)	30-day regional skew (6)
β_0	-0.7340	-0.6901	-0.5872	-0.6445	-0.6322
β_1	0.6778	0.6764	0.5822	0.5375	0.4277
AVP	0.0485	0.0576	0.0490	0.0521	0.0615

Table 22. Regional skew values

Location (1)	Elevation (ft) (2)	Peak flow regional skew (3)	1-day regional skew (4)	3-day regional skew (5)	7-day regional skew (6)	15-day regional skew (7)	30-day regional skew (8)
Farmington Reservoir	621.82	-0.608	-0.734	-0.690	-0.587	-0.644	-0.632
Farmington, CA	605.62	—	-0.734	-0.690	-0.587	-0.644	-0.632

Fitting the curves

As a first step, the curves were fitted using a straightforward *Bulletin 17B* procedure in which all data points were included in the analysis and low outliers were identified by the *Bulletin 17B* outlier test and the station statistics appropriately adjusted. This includes weighting the station skew and regional skew values by the inverse of their associated errors. This weighting procedure is included in *Bulletin 17B* and the weighted skew is automatically calculated by PeakfqSA, which we used here.

We found that this initial fitting of the frequency curves: (1) was sensitive to low flow values, and (2) the 1-day and 3-day flow quantiles for p= 0.01 and p=0.005 annual exceedence probabilities were uncharacteristically large on a flow-per-square mile basis.

We then refitted the frequency curves manually setting the low outlier thresholds for each duration. Specifically, we set these thresholds consistent with those used in the Comp Study. These low outlier thresholds are shown in Table 23 and Table 24.

We then reviewed the curves for appropriateness and consistency. We found the frequency curves on Littlejohn Creek were consistent between durations at each location for the frequencies of interest. However, at Farmington Reservoir the curves associated with the 3-day and 7-day volumes “crossed” for annual exceedence probabilities less than approximately p=0.95. We therefore adjusted the 1-day and 3-day standard deviations consistent with guidance specified in EM 1110-2-1415 (USACE 1993). Specifically, we fit a line to the pairs of mean of the logs and standard deviation of the logs by duration using least squares regression through the data point associated with the peak flow-frequency curve. This relation is shown in Figure 21. We then set the standard deviation of the 1-day and 3-day volumes equal to that specified by this regression. We then reviewed these curves and found that they do not "cross," as would be expected.

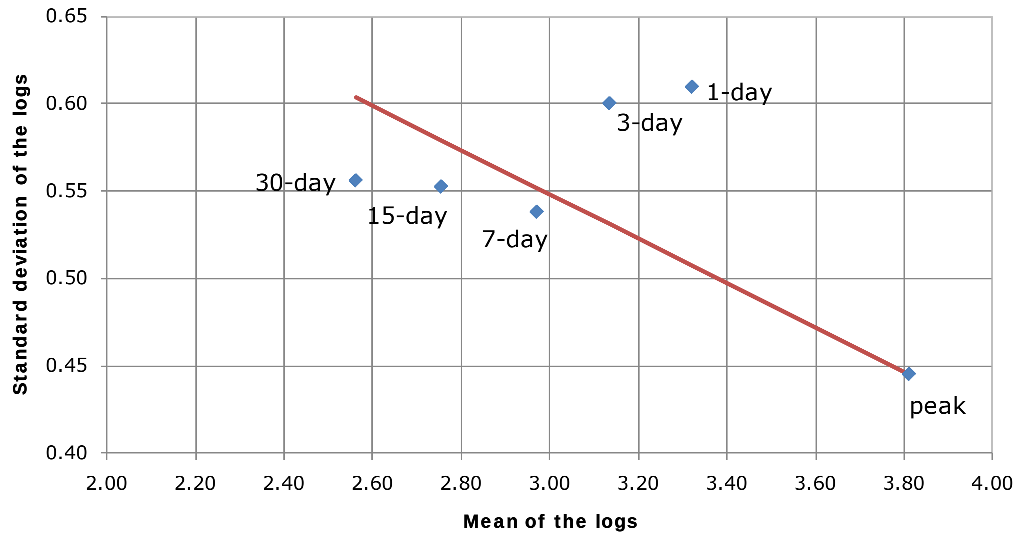


Figure 21. Relationship used to adjust standard deviations at Farmington Reservoir

In addition, we found in our review of the curves at Farmington, CA, that the curve associated with the 30-day volume is inconsistent with the 30-day curve associated with the upstream analysis location at Farmington Reservoir. We therefore set the standard deviation of the 30-day curve at Farmington, CA, equal that of the 30-day curve at Farmington Reservoir. This is consistent with Corps guidance in EM 1110-2-14-15 (USACE 1993). We then reviewed these curves and found that they do not "cross," and flow quantiles for a given duration at the downstream location are greater than those of the upstream location, as would be expected.

As a comparison, we considered the volume-frequency curves developed for Farmington Reservoir in the Comp Study (USACE 2002). The annual maximum series in the Comp Study ended in 1998.

Results

The final parameters and statistics used to fit LPIII distributions to develop the unregulated frequency curves at Farmington Reservoir (shown in Figure 9) are shown in Table 23.

The final parameters and statistics used to fit LPIII distributions to develop the unregulated frequency curves at Farmington, CA, (shown in Figure 10) are shown in Table 24.

Table 23. Unregulated frequency curves parameters and statistics: Farmington Reservoir

Statistic (1)	Peak flows (2)	1-day volumes (3)	3-day volumes (4)	7-day volumes (5)	15-day volumes (6)	30-day volumes (7)
Station mean[1]	3.810	3.301	3.114	2.948	2.733	2.540
Station standard deviation[1]	0.449	0.668	0.661	0.601	0.612	0.615
Station skew[1]	-0.978	-1.410	-1.410	-1.410	-1.410	-1.410
Station skew associated MSE[2]	0.370	0.276	0.275	0.274	0.274	0.273
Regional skew[3]	-0.608	-0.734	-0.690	-0.587	-0.644	-0.632
Regional skew associated AVP[4]	0.140	0.049	0.058	0.049	0.052	0.062
Adopted mean[5]	3.811	3.321	3.135	2.970	2.754	2.561
Standard deviation[5]	0.445	0.610	0.601	0.538	0.553	0.556
Adopted standard deviation	0.445	0.507	0.531	0.538	0.553	0.556
Weighted skew[5,6]	-0.692	-0.858	-0.812	-0.675	-0.733	-0.721
Number of systematic events	34	58	58	58	58	58
Number of high outliers	0	0	0	0	0	0
Number of EMA iterations	2	1	1	1	1	1
Specified low outlier threshold (cfs)	—	282	201	178	105	71
Number of low outliers	0	8	8	8	8	8
Number of zero events	0	0	0	0	0	0
Number of missing events	19	0	0	0	0	0
Number of EMA censored observations	1	8	8	8	8	8
Corresponding censored events[7]	1). 1977	1.) 1977 2.) 1976 3.) 1990 4.) 1989 5.) 1988 6.) 1961 7.) 2003 8.) 1994	1.) 1977 2.) 1976 3.) 1990 4.) 1988 5.) 1989 6.) 1961 7.) 1994 8.) 2003	1.) 1977 2.) 1976 3.) 1989 4.) 1988 5.) 1990 6.) 1961 7.) 1994 8.) 2003	1.) 1977 2.) 1976 3.) 1988 4.) 1989 5.) 1990 6.) 1961 7.) 1994 8.) 2003	1.) 1977 2.) 1976 3.) 1988 4.) 1989 5.) 1990 6.) 1961 7.) 1994 8.) 2003
Record length	53	58	58	58	58	58

Notes:

1. Statistic calculated using the series of logarithmic transforms and EMA without regional skew; rounded to nearest thousandth.
2. Mean square error; rounded to nearest thousandth.
3. Regional skew values calculated using relationships developed by the USGS; rounded to nearest thousandth.
4. Average variance of prediction, analogous to MSE; rounded to nearest thousandth.
5. Statistic calculated using the series of logarithmic transforms and EMA with regional skew; rounded to nearest thousandth.
6. Skew value calculated by weighting the station and regional skew values inversely proportional to their associated errors: (MSE and AVP) and EMA; rounded to nearest thousandth.
7. Events are listed by water year in order of increasing flow or volume.

Table 24. Unregulated frequency curves parameters and statistics: Farmington, CA

Statistic (1)	1-day volumes (2)	3-day volumes (3)	7-day volumes (4)	15-day volumes (5)	30-day volumes (6)
Station mean[1]	3.339	3.169	2.992	2.797	2.628
Station standard deviation[1]	0.621	0.593	0.579	0.573	0.539
Station skew[1]	-1.410	-1.410	-1.410	-1.410	-1.268
Station skew associated MSE[2]	0.278	0.276	0.276	0.276	0.251
Regional skew[3]	-0.734	-0.690	-0.587	-0.644	-0.632
Regional skew associated AVP[4]	0.049	0.058	0.049	0.052	0.062
Adopted mean[5]	3.356	3.186	3.011	2.815	2.639
Standard deviation[5]	0.573	0.545	0.525	0.523	0.507
Adopted standard deviation	0.573	0.545	0.525	0.523	0.556
Weighted skew[5,6]	-0.849	-0.786	-0.670	-0.722	-0.695
Number of systematic events	58	58	58	58	58
Number of high outliers	0	0	0	0	0
Number of EMA iterations	1	1	1	1	1
Specified low outlier threshold (cfs)	307	254	178	117	82
Number of low outliers	7	7	7	7	7
Number of zero events	0	0	0	0	0
Number of missing events	0	0	0	0	0
Number of EMA censored observations	7	7	7	7	6
Corresponding censored events[7]	1.) 1976 2.) 1977 3.) 1961 4.) 1989 5.) 1990 6.) 1988 7.) 2003	1.) 1976 2.) 1977 3.) 1961 4.) 1989 5.) 1990 6.) 1988 7.) 2003	1.) 1976 2.) 1977 3.) 1961 4.) 1989 5.) 1990 6.) 1988 7.) 2003	1.) 1976 2.) 1961 3.) 1977 4.) 1990 5.) 1989 6.) 1988 7.) 2003	1.) 1961 2.) 1989 3.) 1990 4.) 1977 5.) 1989 6.) 2003
Record length	58	58	58	58	58

Notes:

1. Statistic calculated using the series of logarithmic transforms and EMA without regional skew; rounded to nearest thousandth.
2. Mean square error; rounded to nearest thousandth.
3. Regional skew values calculated using relationships developed by the USGS; rounded to nearest thousandth.
4. Average variance of prediction, analogous to MSE; rounded to nearest thousandth.
5. Statistic calculated using the series of logarithmic transforms and EMA with regional skew; rounded to nearest thousandth.
6. Skew value calculated by weighting the station and regional skew values inversely proportional to their associated errors: (MSE and AVP) and EMA; rounded to nearest thousandth.
7. Events are listed by water year in order of increasing volume.

Attachment 5: Unregulated-regulated flow transforms and critical duration assessment

Fit unregulated-regulated flow transforms

We developed the unregulated-regulated flow transforms for the 2 analysis locations by fitting transform curves through data pairs from the event maxima datasets. Specifically, we fitted transforms to pairs of unregulated volumes (as average flows) and regulated peak flows. For this analysis, we used unregulated volumes associated with the 1-, 1.5-, 2-, 2.5-, 3-, 3.5-, 4-, 4.5-, 5-, 6-, 7-, 10-, 15-, and 30-day durations. We fitted these curves to the data pairs of historical and scaled events using the robust locally weighted scatterplot smoothing (LOWESS) regression technique. (The LOWESS procedure is detailed in the *Technical procedure document*.)

Here, we used the LOWESS algorithm developed by William Cleveland (Cleveland 1985). We complied an executable of the algorithm, implemented in Fortran. This executable was tested using example data included in the Fortran file.

We used an iterative process to fit these transforms. Specifically we:

- Fitted a candidate transform using the LOWESS regression technique.
- Calculated the mean squared error (MSE) associated with the candidate transform.
- Modified the LOWESS parameters using guidance provided in the literature (Bradley and Potter 2004, Cleveland 1979).
- Fitted another candidate transform and calculated the associated MSE.
- Compared this new transform to the old transform(s) visually and based on MSE.
- Repeated the previous steps until the parameters resulting in the best fit, as determined visually and based on MSE, were identified.

Determine critical duration

For a regulated system, the critical duration is the unregulated flow duration-frequency curve that best characterizes the peak regulated flow-frequency curve at a downstream point. To determine critical duration for each location, we:

- Fitted flow transforms to the event maxima datasets as detailed in the previous subsection.
- Applied these flow transforms to develop hypothetical regulated flow-frequency curves.
- Identified the duration of the unregulated annual maximum series that estimates the largest flow for each probability of interest, as shown in column 1 of Table 25. Here, we considered2 criteria: (1) the "goodness of fit" of each transform, and (2) which duration estimates the greater peak regulated flows

Table 25. Synthesis of information used to determine critical duration

Annual exceedence probability (1)	Unregulated flow duration (in days) that estimates the largest flow quantile at Farmington Reservoir (2)	Farmington, CA[1] (3)
0.500	15	10
0.200	2.5	3.5
0.100	2.5	3.5
0.050	15	1
0.020	15	10
0.010	15	10
0.005	10	10
0.002	10	10

Notes:

1. For Farmington, CA, we list the duration equal or less than 15 days that estimates the largest flow.

After considering all the durations noted above, for Farmington Reservoir we focused on durations of 15 days or less because: (1) the typical unregulated inflow event duration is less than 15 days, and (2) the flow transforms for durations of 15 days or less better fit the event maxima data pairs based on MSE and visual inspection. In addition, the scaled historical event unregulated volumes associated with the longer durations tend to include volumes of additional flood waves after the peak reservoir release. These later flood waves do not contribute to the inflow volumes that drive the reservoir releases, unlike multiple flood waves prior to the peak reservoir releases that are considered. Here, we defined a flood event as the time from when the pool elevation rises from and returns to the top of conversation pool (bottom of flood control pool). For Farmington, CA, we looked at durations equal or less than the critical duration at Farmington Reservoir because the addition of unregulated local flows will not cause the critical duration to increase.

In selection of the critical duration, we gave more weight to the durations that estimated the largest flow quantiles for the p=0.01, p=0.005, and p=0.002 annual exceedence events. We used these probabilities because Farmington Reservoir has large flood storage volume, and regulated peak flows associated with more common events are driven by local flow peaks, not reservoir inflow volumes for a given duration.

From this analysis we determined that the critical duration at Farmington Reservoir and at Farmington, CA, is 10 days. Thus, the appropriate unregulated-regulated flow transforms used in this analysis were associated with this duration. The critical duration associated with the downstream operation point is shorter than that of the reservoir because of the effects of local flow.

As a "reality check" on our critical duration values, we simulated events, with the HEC-ResSim model, that corresponded to specific volumes associated with a given duration and annual exceedence probability. This is an alternative option for assessing critical duration as detailed in Attachment F of the *Technical Procedures document* as "Method 2: Limited sample, specific volume-duration event scaling." For this check, we scaled reservoir inflows for

4 event patterns (1969, 1986, 1998, and 2006) to the 1-, 3-, 7-, and 10-day unregulated flows for the p=0.01, and p=0.005 annual exceedence probabilities. We found: (1) the resulting regulated peaks sensitive to hydrograph shape, and (2) the scaling to the 1-, 3-, and 10-day durations estimated largest regulated peak flows. These results are consistent with the adopted critical duration values for the 2 analysis locations.

Review and adopt transforms

After determining the critical duration associated with each analysis location, we reviewed the unregulated-regulated flow transforms initially fitted with the LOWESS procedure to: (1) check for appropriateness, and (2) identify the need for adjustments, if any. As part of this review we:

- Compared event hydrographs of the simulated events that correspond to the transitional areas of the transform (i.e., where the objective peak flows are being constrained, or where peak releases become larger than the objective).
- Fitted additional transforms omitting scaled historical events with scale factors of 2 or less.
- Identified and compared the unregulated volumes that define the "break points" where large floods-of-record and their scaled versions were not controlled by the reservoir because of (1) lack of storage capacity, or (2) local flows larger than the channel capacity.
- Split the unregulated-regulated flow transform initially fitted with LOWESS into 2 ranges using this break point.
- Calculated the MSE for these 2 ranges for each initially fitted LOWESS curve.
- Identified which LOWESS curves have the least MSE for each range.

For both analysis locations, we found that the LOWESS fitted curves with a smoothing coefficient of 0.2 had lowest MSE for ranges of unregulated 10-day volumes both larger and smaller than that associated with the "break point."

We adjusted the unregulated-regulated flow transform at Farmington Reservoir based on our review of selected historical events and sensitivity analysis of the LOWESS fitting of the transform. Specifically, we refined the transform using linearly interpolation for regulated peak flows between 2,000 cfs and approximately 3,100 cfs.

As a final check, we re-applied the transform to compute the associated regulated flow quantiles. We compared these quantiles to those associated with the original fit, and those associated with the candidate transforms for the other unregulated volumes. For both locations, we computed: (1) small decreases for quantiles with annual exceedence probability equal or greater p=0.05, and (2) no change in quantiles with annual exceedence probability equal or less p=0.01.

Based on this review, we adopted flow transforms for Farmington Reservoir and Farmington, CA, shown in Figure 22 and Figure 23. The tabulated curves are in an MS Excel file on DVD with the original report.

In Figure 22 and Figure 23 we show the unregulated-regulated flow transforms in black dashes, the floods-of-record event maxima in red squares, the historical scaled event maxima in green triangles, and the initial LOWESS fitted flow transforms in blue for comparison. We also show in grey in Figure 22 and Figure 23 the corresponding unregulated volume-duration quantiles for annual exceedence probabilities of interest.

We show in Table 26 and Table 27 the parameters we used to fit these transforms and the resulting mean square errors. Highlighted in grey in Table 26 and Table 27 are the LOWESS fitted curves with smoothing coefficients listed in column 1 used in fitting the final unregulated-regulated flow transforms over the ranges specified in columns 4 and 5.

Table 26. LOWESS parameters and resulting errors for fitting of unregulated-regulated flow transforms: Farmington Reservoir

Smoothing coefficient[1] (1)	Number of iterations[2] (2)	Delta[3] (3)	Minimum threshold (1,000 cfs) (4)	Maximum threshold (1,000 cfs) (5)	Total number of data pairs (6)	MSE[4] (7)
0.2	2	0	0.5	10	186	964,227
			0.5	5	120	189,155
			5	10	66	2,373,450
Adopted transform			**0.5**	**10**	**—**	**973,765**

Notes:

1. The fraction of points used to calculate each point of the flow transform.
2. The number of iterations used in calculating the robust fitted curve. A value of 2 returns a robust fit.
3. Delta is a nonnegative value used by the program we used to compute the LOWESS algorithm to "save intermediate computations," and reduces computation time for large datasets. In this study the datasets are small, and thus this was set to 0.
4. Mean square error over the range of interest defined by the minimum and maximum thresholds listed in columns 4 and 5.

Table 27. LOWESS parameters and resulting errors for initial fitting of unregulated-regulated flow transforms: Farmington, CA

Smoothing coefficient[1] (1)	Number of iterations[2] (2)	Delta[3] (3)	Minimum threshold (1,000 cfs) (4)	Maximum threshold (1,000 cfs) (5)	Total number of data pairs (6)	MSE[4] (7)
0.2	2	0	0.5	10.5	188	1,366,865
			0.5	5	117	335,543
			5	10.5	71	3,066,368
Adopted transform			**0.5**	**10.5**	**—**	1,385,920

Notes:

1. The fraction of points used to calculate each point of the flow transform.
2. The number of iterations used in calculating the robust fitted curve. A value of 2 returns a robust fit.
3. Delta is a nonnegative value used by the program we used to compute the LOWESS algorithm to "save intermediate computations," and reduces computation time for large datasets. In this study the datasets are small, and thus this was set to 0.
4. Mean square error over the range of interest defined by the minimum and maximum thresholds listed in columns 4 and 5.

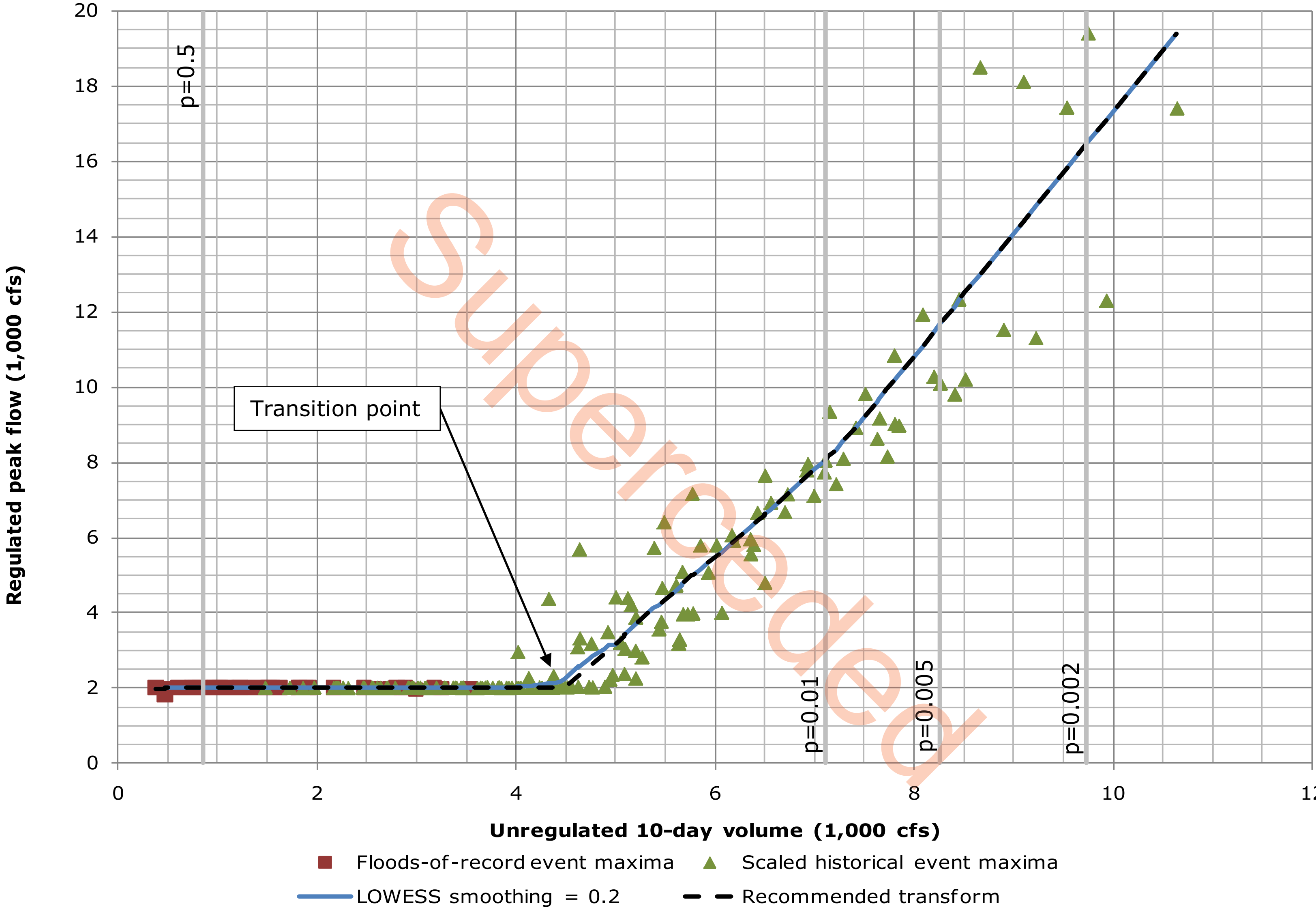


Figure 22. Unregulated-regulated flow transform and LOWESS fitted curves: Farmington Reservoir

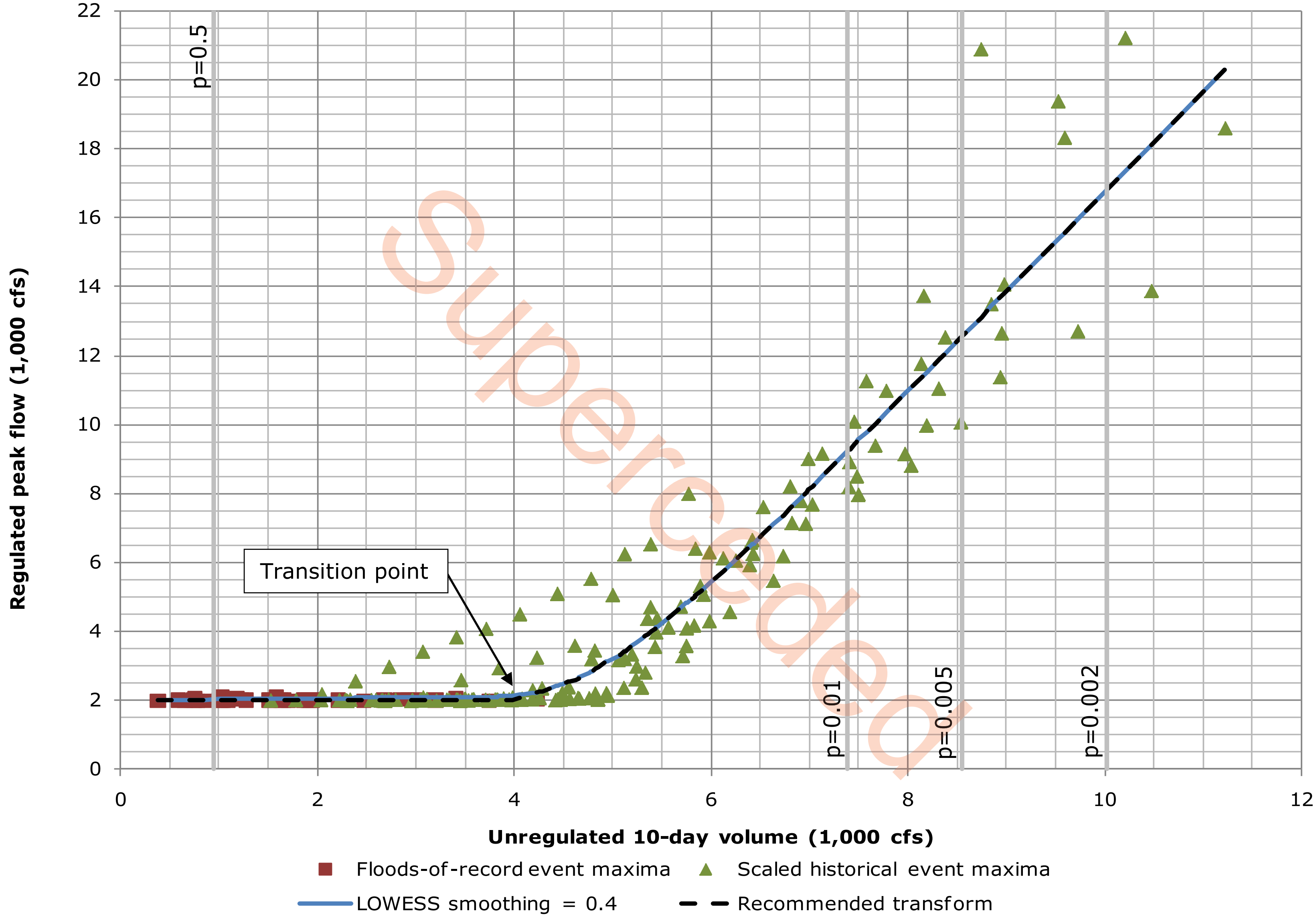


Figure 23. Unregulated-regulated flow transform and LOWESS fitted curve: Farmington, CA

Attachment 6: Family of regulated characteristic curves

Fit the characteristic curves

We used the families of regulated characteristic curves to relate a given regulated peak flow to likely associated regulated volumes at each analysis location. We developed the families of regulated characteristic curves for Farmington Reservoir and at Farmington, CA, by fitting transform curves through the pairs of event regulated volumes, as average flows, and regulated peak flows. The fitting is similar to how we developed the unregulated-regulated transforms detailed in Attachment 5. The datasets we used include both historical and scaled events to define the extreme ends of the flow transform curve.

We initially fitted these curves to the data pairs of historical and scaled events using the LOWESS regression technique and parameters shown in Table 28 and Table 29 for Farmington Reservoir and at Farmington, CA. In this initial fitting we used the entire event maxima dataset for the given analysis location. Because the flows of interest correspond to events equal or larger than the p=0.5 event, but less than or equal to the p=0.002 event, we truncated the datasets of event pairs to the minimum and maximum regulated flow thresholds specified in columns 5 and 6 of Table 28 and Table 29 for selection of the appropriate LOWESS smoothing coefficient to use in developing the characteristic curves. Highlighted in grey in Table 28 and Table 29 are the LOWESS fitted curves with smoothing coefficients listed in column 2 used in fitting the final characteristic curves for the duration specified in column 1 over the range with minimum and maximum flow thresholds specified in columns 5 and 6.

Review and adopt the characteristic curves

We reviewed and adjusted the curves initially fitted with the LOWESS procedure using the same process detailed for fitting the unregulated-regulated flow transforms. Here, the only difference is that the "break point" is defined by the downstream objective flow (2,000 cfs). Thus the mean square errors in the LOWESS fitted curves were compared over these 2 ranges for each characteristic curve.

From this review we found:

- The families of regulated characteristic curves were consistent between durations at both locations. That is, they do not cross.
- The fit of the curves at Farmington, CA, was sensitive to large diversions from Duck Creek such as those in the 1995 event and its corresponding scaled events.
- The characteristic volume at Farmington, CA, for a given annual exceedence and duration may be less than the characteristic volume associated with Farmington Reservoir for the same annual exceedence probability because this effect of diversions. However, the regulated peak flow at Farmington, CA, is always equal or larger than the peak at Farmington Reservoir for the same exceedence probability.

Based on this review, we adopted the adjusted families of curves.

We show in Figure 24 through Figure 28 the regulated characteristic curves corresponding to Farmington Reservoir. In addition, we include tabulations of this family of regulated characteristic curves in an MS Excel file on the DVD included with the original report.

We show in Figure 29 though Figure 33 regulated characteristic curves corresponding to Farmington, CA. In addition, we include tabulations of this family of regulated characteristic curves in an MS Excel file on the DVD included with the original report.

In Figure 24 through Figure 33 we show the characteristic curves in black, the floods-of-record event maxima in red squares, the historical scaled event maxima in green triangles, and the initial LOWESS fitted flow curves in blue for comparison.

Table 28. LOWESS parameters for fitting the family of regulated characteristic curves and resulting errors: Farmington Reservoir

Duration (days) (1)	Smoothing coefficient[1] (2)	Number of iterations[2] (3)	Delta[3] (4)	Minimum threshold (1,000 cfs) (5)	Maximum threshold (1,000 cfs) (6)	Total number of data pairs (7)	LOWESS curve MSE[4] (8)	Characteristic curve MSE (9)
1	0.2	2	0	2	16.5	182	7,606	7,687
3							99,693	100,058
7							270,829	279,316
15							276,837	339,035
30							183,572	290,625

Notes:

1. The fraction of points used to calculate each point of the flow transform.
2. The number of iterations used in calculating the robust fitted curve. A value of 2 returns a robust fit.
3. Delta is a nonnegative value used by the program we used to compute the LOWESS algorithm to "save intermediate computations," and reduces computation time for large datasets. In this study the datasets are small, and thus this was set to 0.
4. Mean square error over the range of interest defined by the minimum and maximum thresholds listed in columns 5 and 6.

Table 29. LOWESS parameters for fitting the family of regulated characteristic curve and resulting errors: Farmington, CA

Duration (days) (1)	Smoothing coefficient[1] (2)	Number of iterations[2] (3)	Delta[3] (4)	Minimum threshold (1,000 cfs) (5)	Maximum threshold (1,000 cfs) (6)	Total number of data pairs (7)	LOWESS curve MSE[4] (8)	Characteristic curve MSE (9)
1	0.2	2	0	2	17	185	83,489	83,473
3							174,784	174,806
7							334,875	334,900
15							303,171	309,865
30							176,684	185,114

Notes:

1. The fraction of points used to calculate each point of the flow transform.
2. The number of iterations used in calculating the robust fitted curve. A value of 2 returns a robust fit.
3. Delta is a nonnegative value used by the program we used to compute the LOWESS algorithm to "save intermediate computations," and reduces computation time for large datasets. In this study the datasets are small, and thus this was set to 0.
4. Mean square error over the range of interest defined by the minimum and maximum thresholds listed in columns 5 and 6.

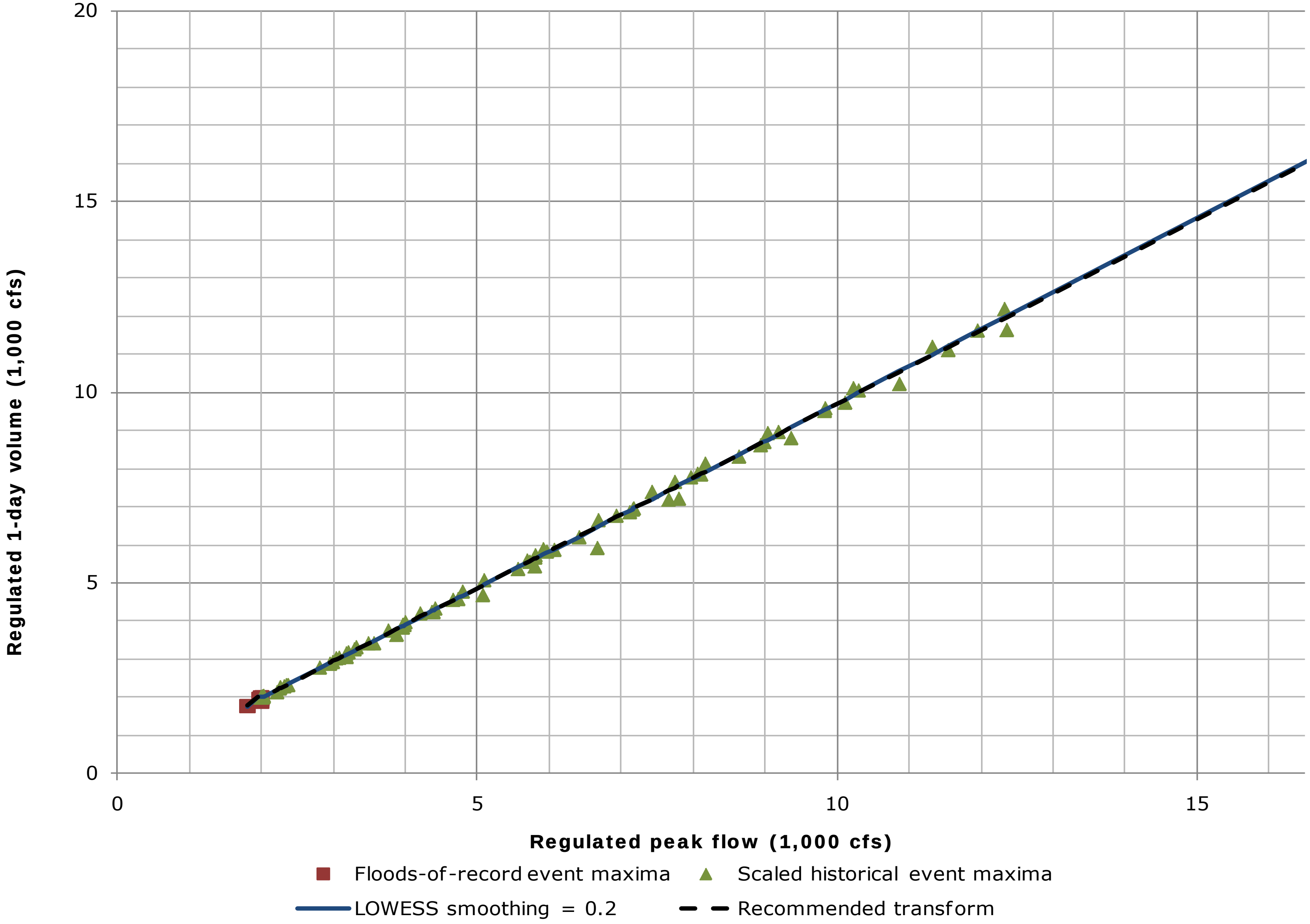


Figure 24. Farmington Reservoir regulated characteristic curve: 1-day duration

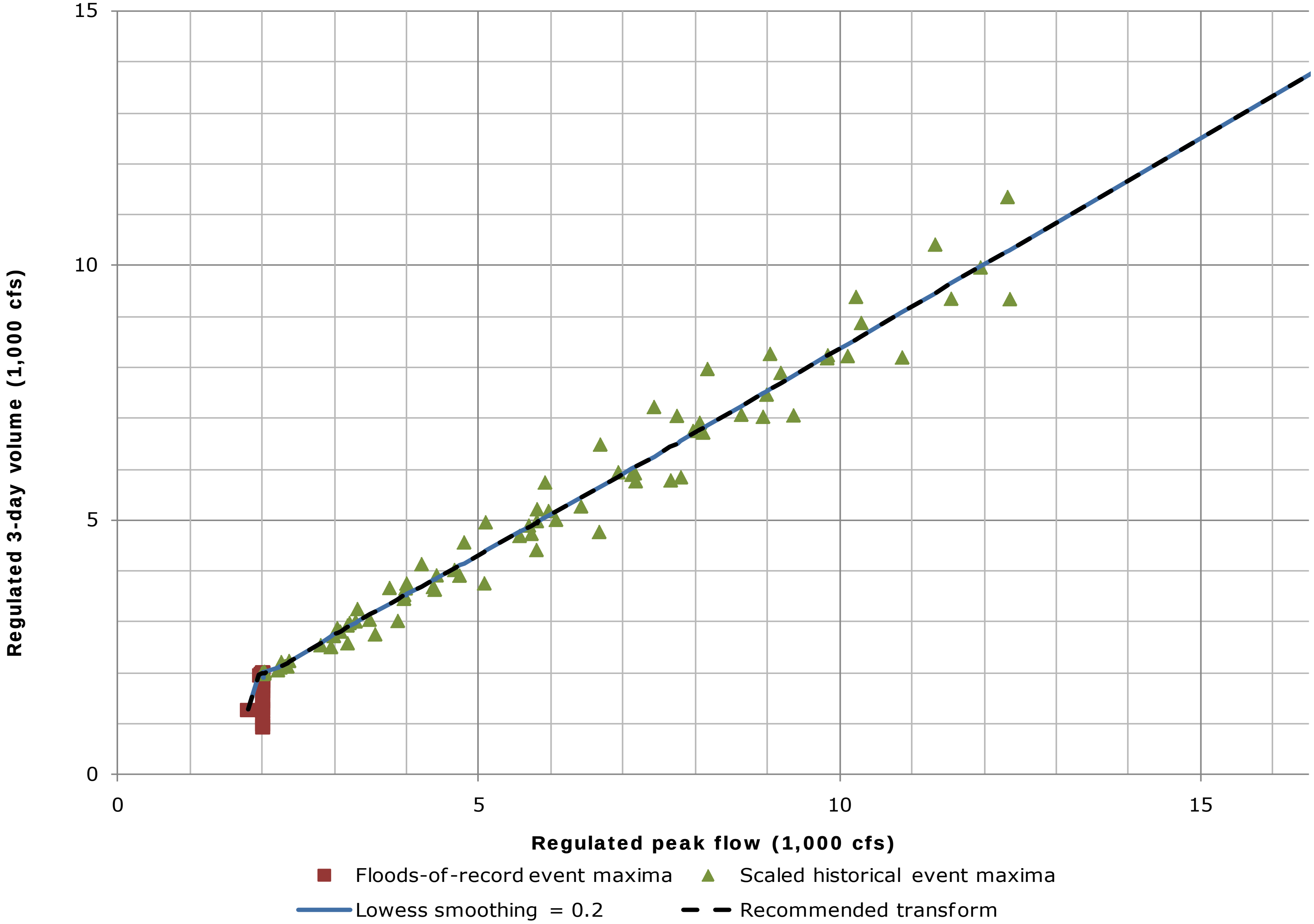


Figure 25. Farmington Reservoir regulated characteristic curve: 3-day duration

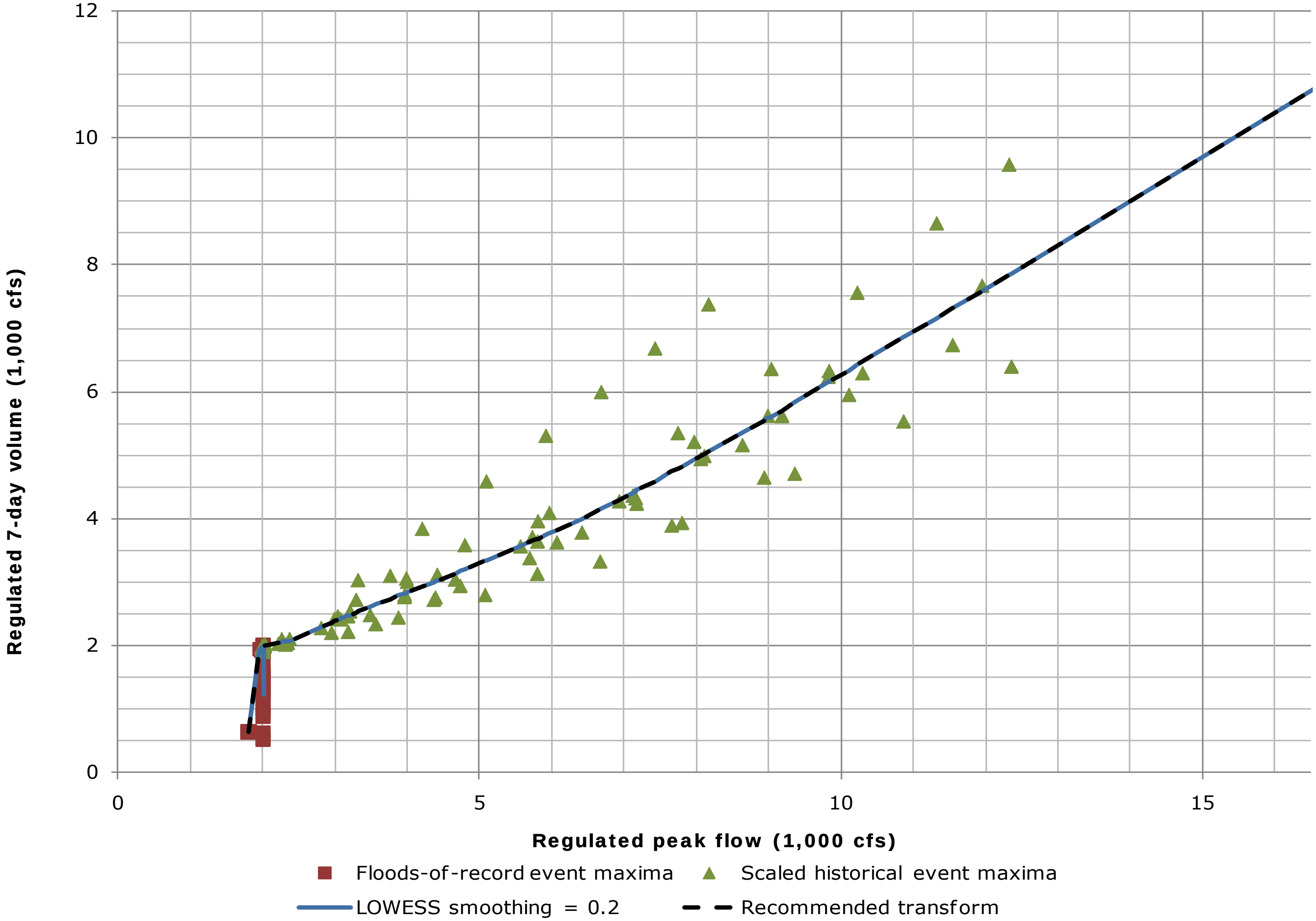


Figure 26. Farmington Reservoir regulated characteristic curve: 7-day duration

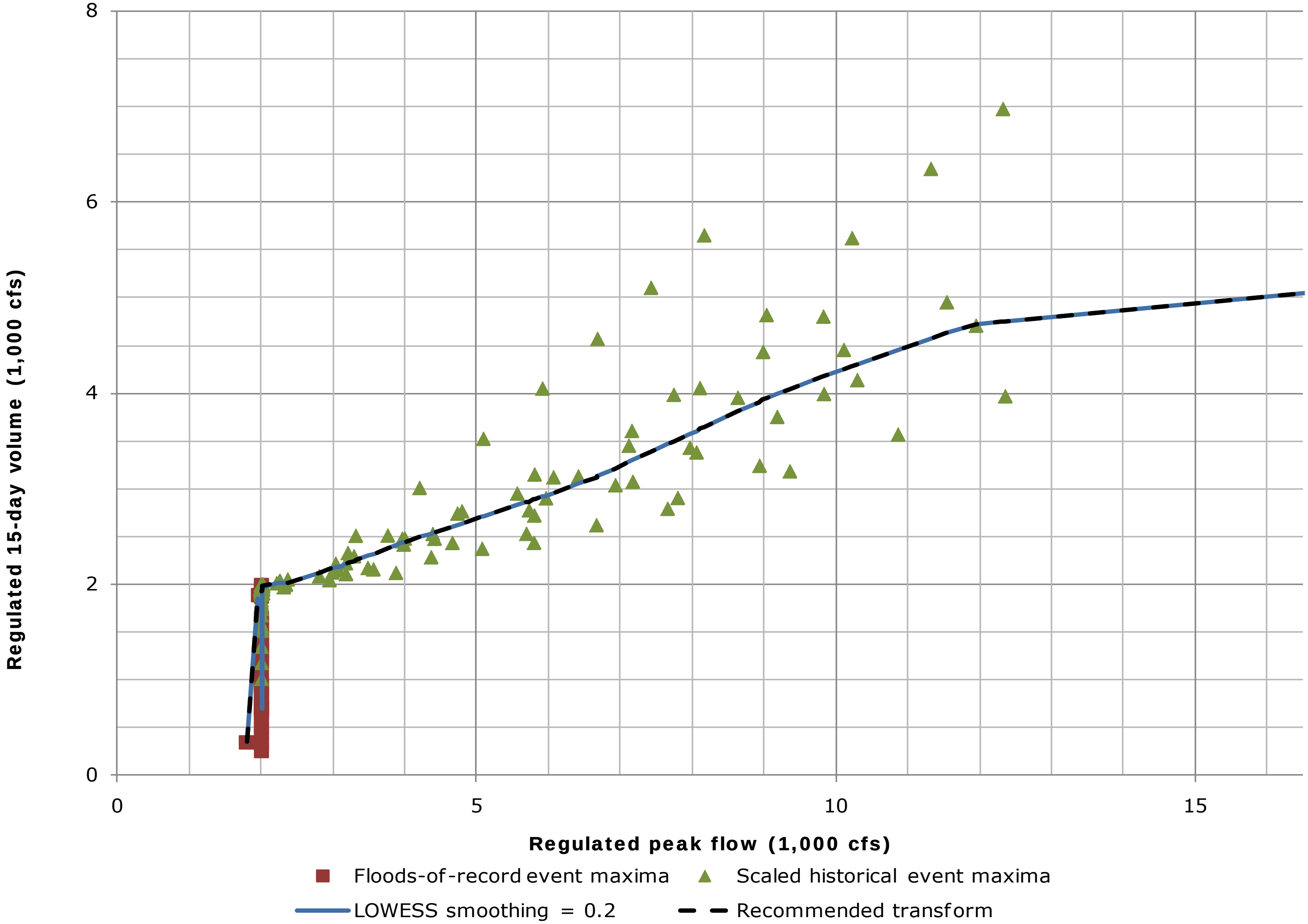


Figure 27. Farmington Reservoir regulated characteristic curve: 15-day duration

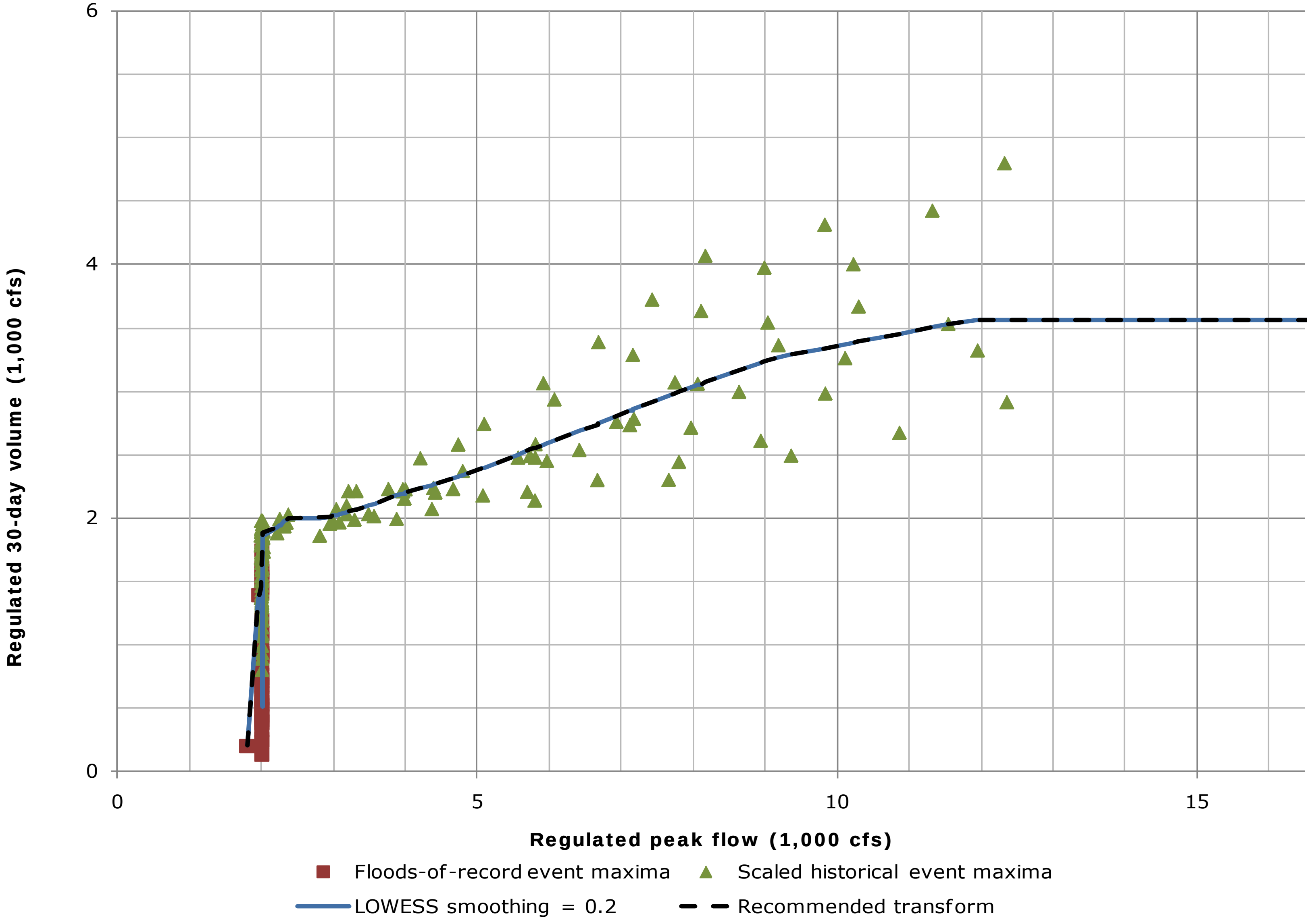


Figure 28. Farmington Reservoir regulated characteristic curve: 30-day duration

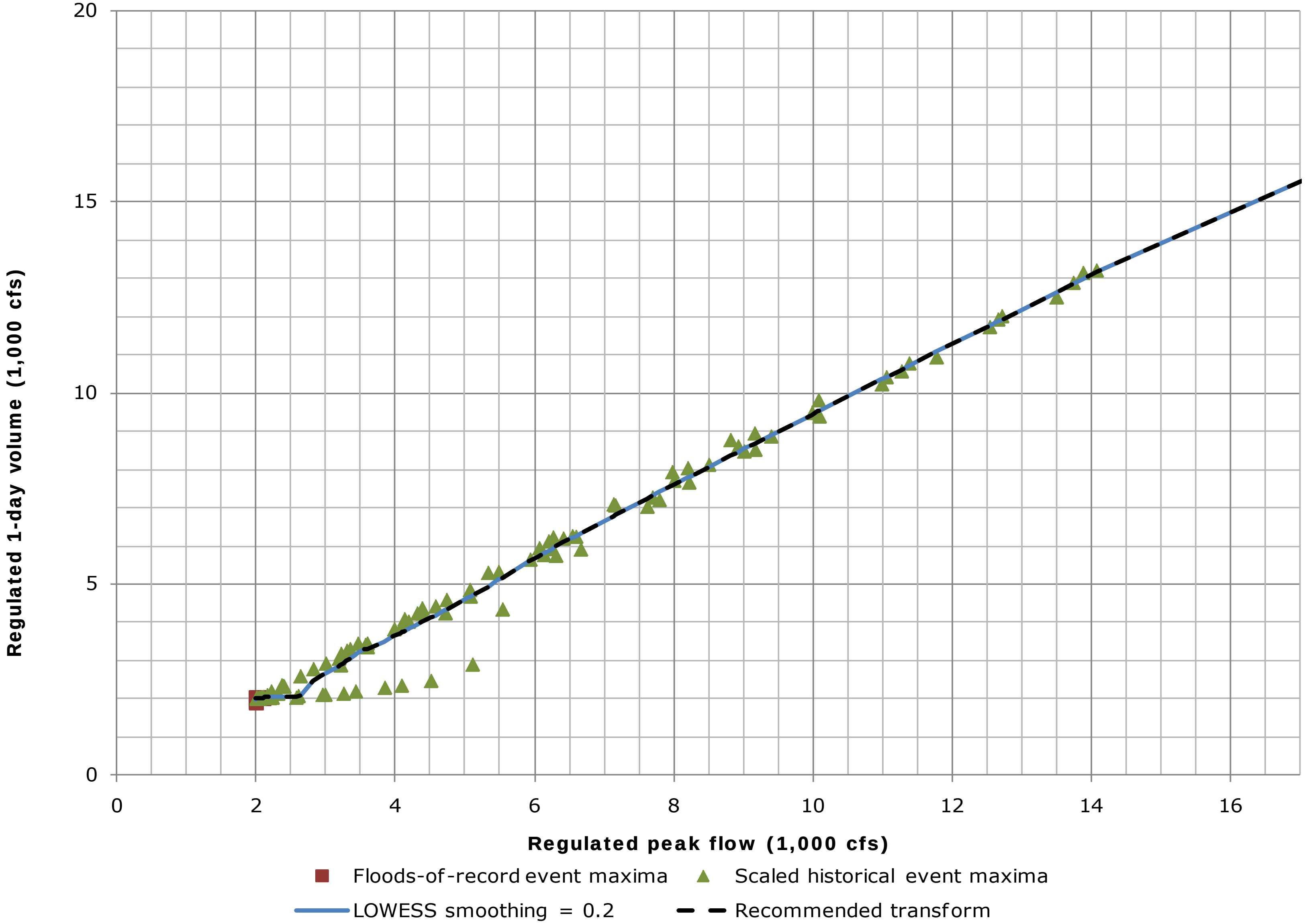


Figure 29. Littlejohn Creek at Farmington, CA, regulated characteristic curve: 1-day duration

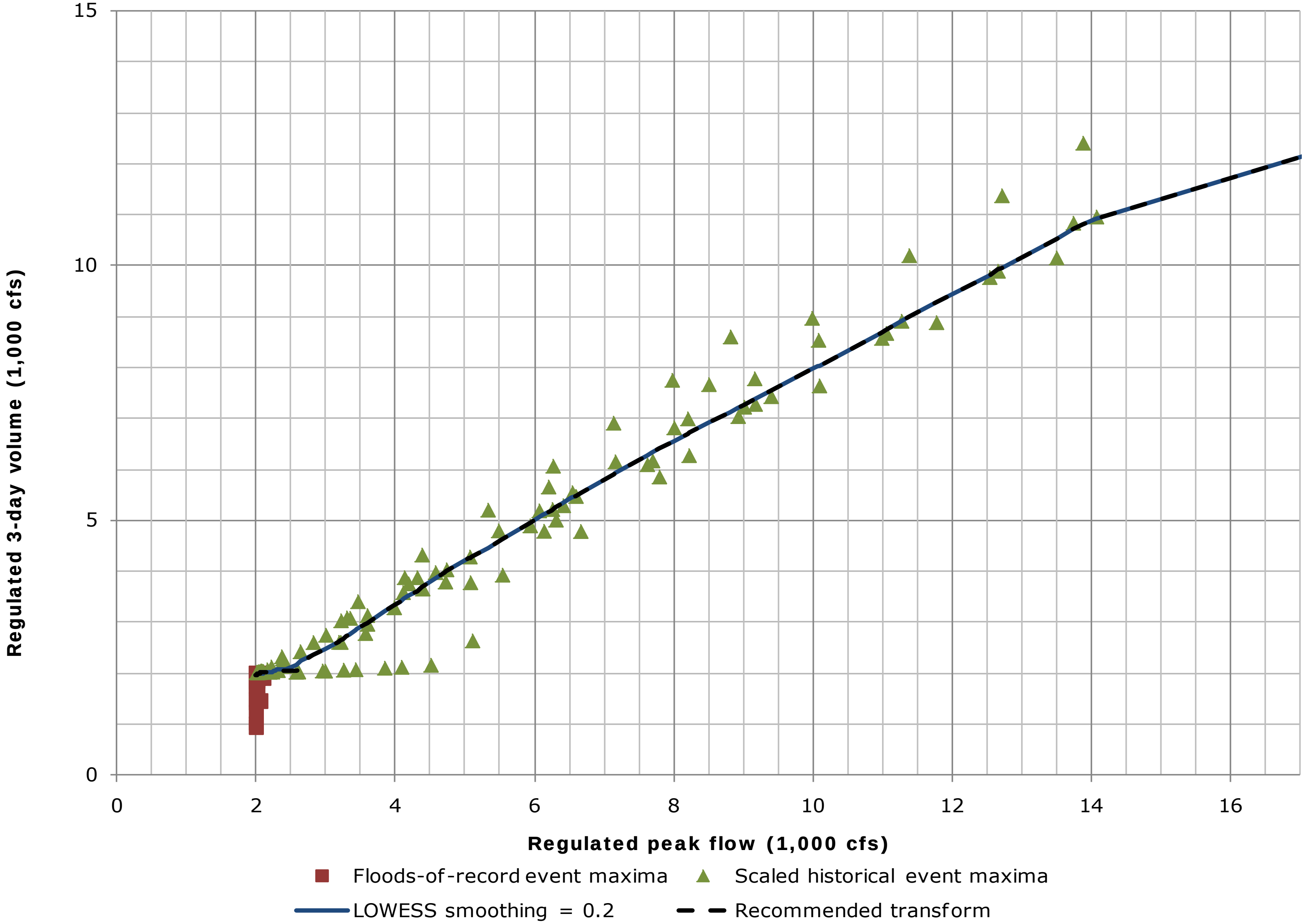


Figure 30. Littlejohn Creek at Farmington, CA, regulated characteristic curve: 3-day duration

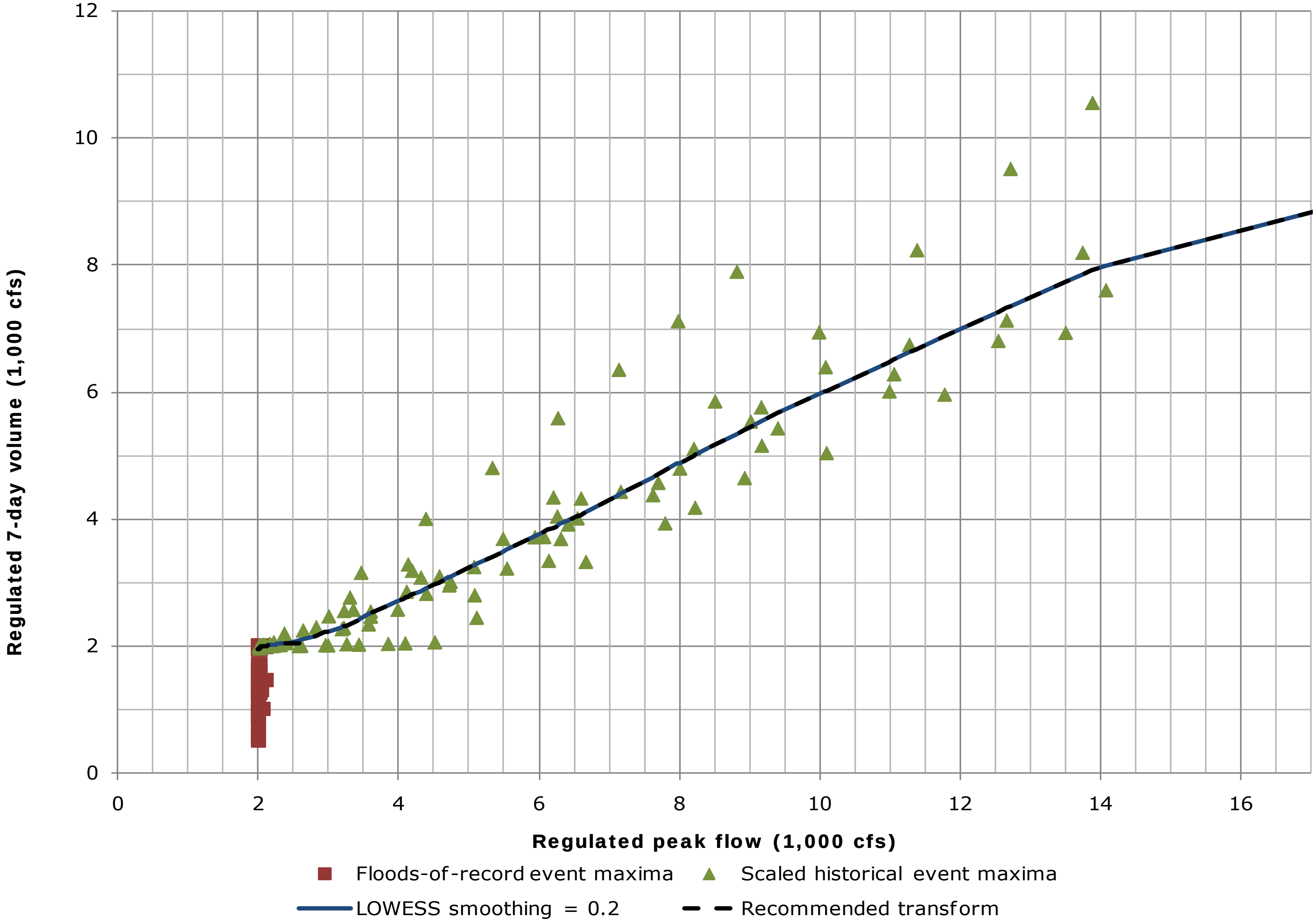


Figure 31. Littlejohn Creek at Farmington, CA, regulated characteristic curve: 7-day duration

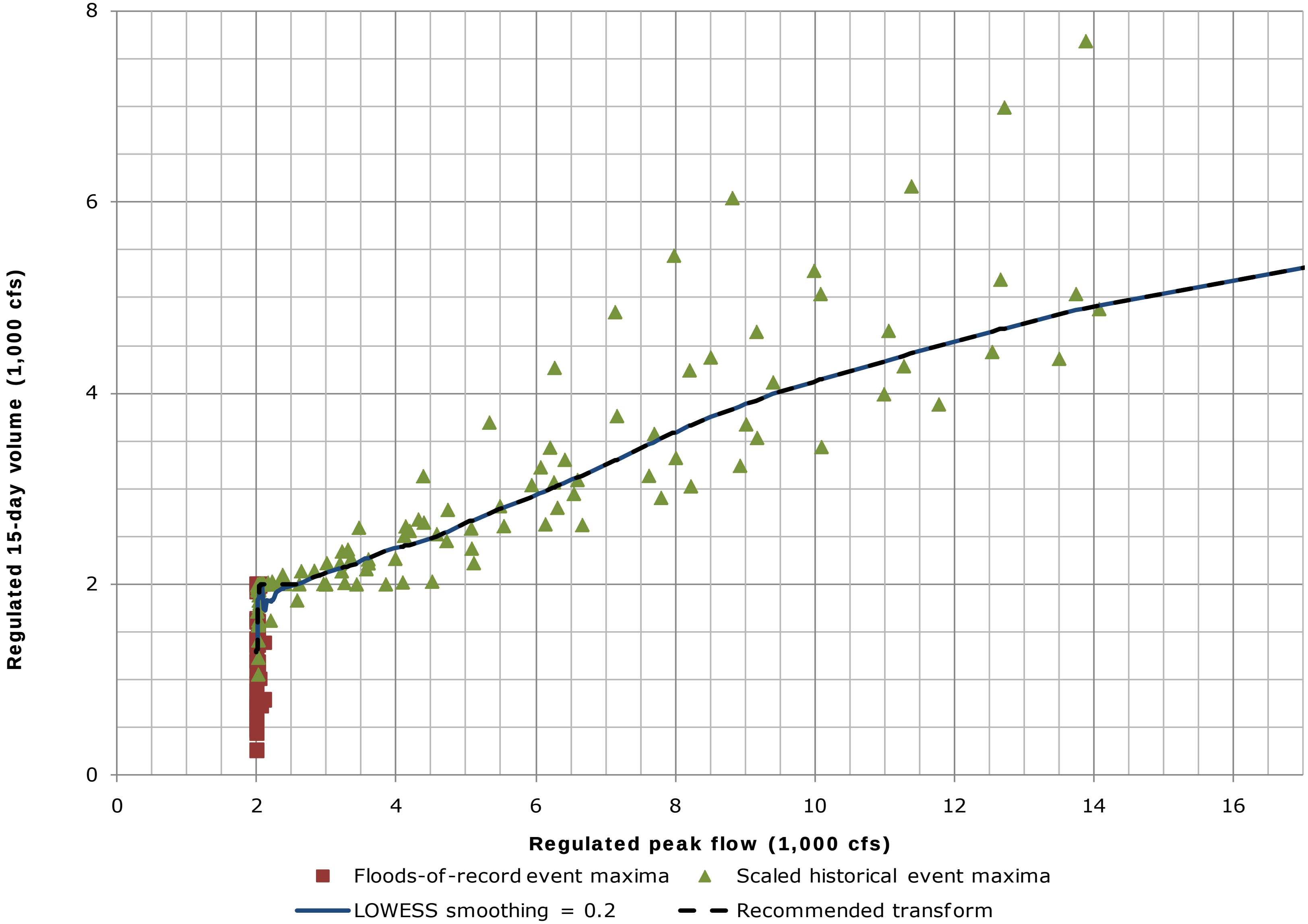


Figure 32. Littlejohn Creek at Farmington, CA, regulated characteristic curve: 15-day duration

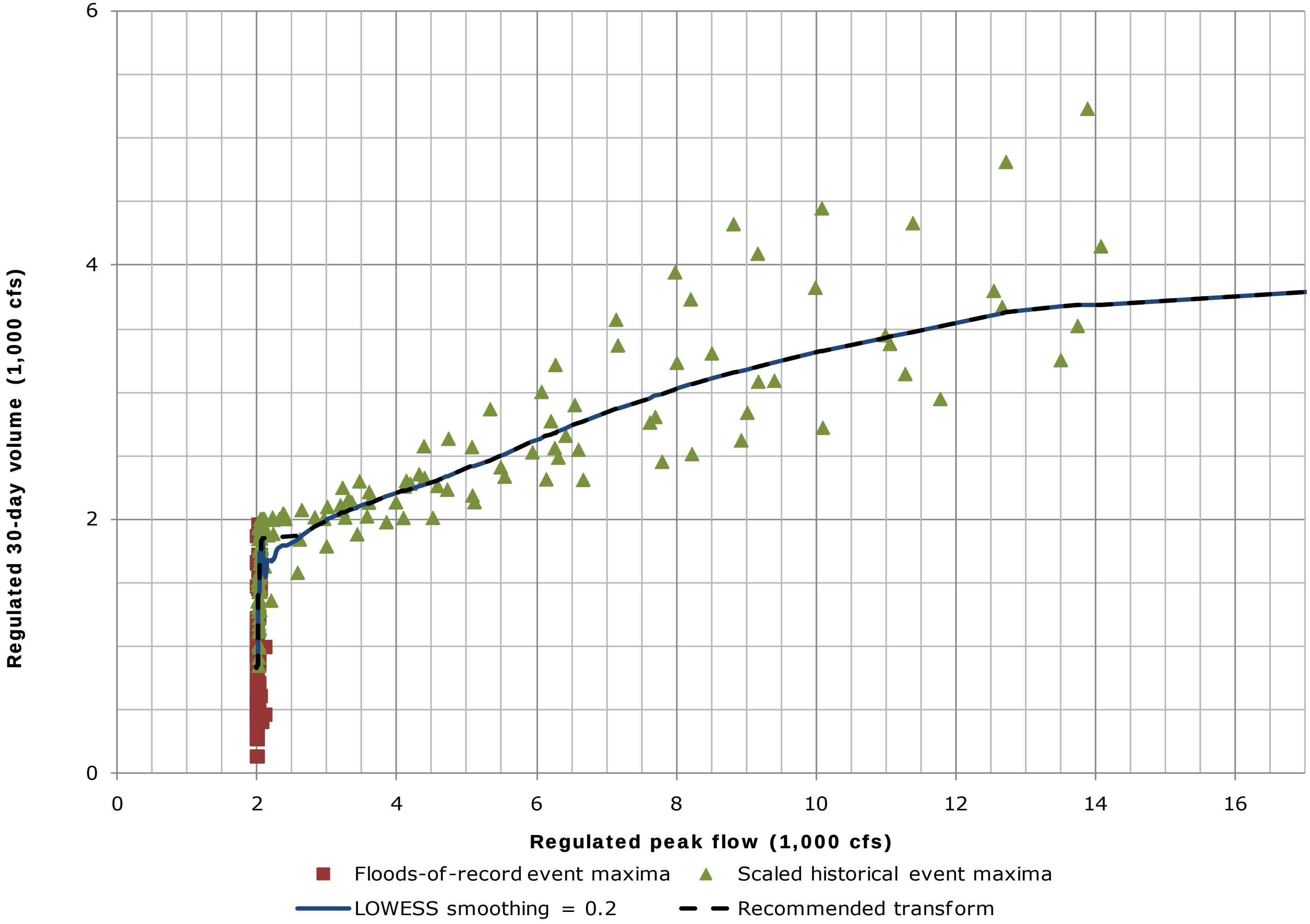


Figure 33. Littlejohn Creek at Farmington, CA, regulated characteristic curve: 30-day duration

Attachment 7: Quality control certification

David Ford Consulting Engineers, Inc. completed Task 3, development of flow frequency curves, expected hydrographs, and documentation of procedures for contract W91238-09-D-0004—Lower San Joaquin River Feasibility Study, San Joaquin County, CA including Stockton City and nearby communities.

Notice is hereby given that all quality control activities of the technical memorandum prepared by the firm have been completed, appropriate to the level of risk and complexity inherent in the project, as defined in the Quality Control Plan. Compliance with established policy principles and procedures, utilizing justified and valid assumptions, was verified. This includes review of assumptions; methods, procedures, and material used in the analyses; the appropriateness of data used and level of data obtained; and reasonableness of the results, including whether the product is consistent with law and existing Corps policy.

REGISTERED PROFESSIONAL ENGINEER
DAVID THURMAN FORD
No. 33889
Exp. 6/30/2012
CIVIL
STATE OF CALIFORNIA

3/25/2011

David T. Ford, PhD, PE, D.WRE
President
David Ford Consulting Engineers, Inc.

(date)

Appendix 3

Lower San Joaquin River Feasibility Study

Hydrologic Analysis for Bear Creek, Mosher Slough, Calaveras River watershed below Bellota, and French Camp Slough

US Army Corps
of Engineers.

April 2014

Table of Contents

Attachments

1. Attachment 1: Lower San Joaquin River Feasibility Study, F3 Hydrology Appendix by Peterson, Brustad, Inc. dated July 30, 2012.

References

1. Lower San Joaquin River Feasibility Study, F3 Hydrology Appendix by Peterson, Brustad, Inc. dated July 30, 2012.

1.0 Background

This Appendix covers the hydrologic analysis for Bear Creek, Mosher Slough, Calaveras River watershed below Bellota, and French Camp Slough. Peterson Brustad, Inc. (PBI) studied these watershed areas (Ref 1) while the Sacramento District of the US Army Corps of Engineers (SPK) and David Ford Consulting Engineers (DFC) analyzed: 1) New Hogan Dam down to the downstream control point (Mormon Slough at Bellota) and 2) Farmington Dam down to the downstream control point (Littlejohn Creek at Farmington, Ca). PBI studied the portions of the watershed that required rainfall runoff models due to a lack of sufficient gaged flow data; while SPK and DFC analyzed the largely regulated portions of the study area that could be analyzed via measured flows and reservoir simulation models.

The first part of this appendix describes multiple analyses performed jointly by PBI and SPK after the initial ATR review. These were meant to address address concerns about 1) the calibration of the lower Calaveras River HMS model and 2) the nature of the design storms. The concerns about the storm include: a) the design storm was not balanced to multiple durations (PBI balanced a 1997 pattern hydrograph to only one duration – the 72-hour NOAA14 depth) b) only one areal reduction factor was applied to the storm (72-hour) and c) the adopted storm centering approach for the area downstream of Bellota caused a lack of clarity about what the hydrographs at downstream index points actually represented (i.e. a specific frequency flow or something else). The PBI Report for Bear Creek, Mosher Slough, Calaveras River watershed below Bellota, and French Camp Slough is attached to this Appendix 3 to provide further details on their analysis.

Significant Findings: The results of the new rainfall runoff calibration efforts indicated that the original PBI modeling parameters were appropriate and did not need modification. For the design storm concerns, SPK created a fully balanced design storm for the 1-,3-, 6-, 12-, 24-, 48-, and 72-hour depth durations from NOAA14 using the January 1997 storm pattern. The appropriate HMR 59 areal reduction factors that go along with each duration were applied. The storm centering method was to assume a storm bullseye for the whole drainage area above Mormon Slough at Bellota, with only concurrent and more common frequency rainfall occurring between Bellota and Stockton. As the Bellota hydrograph that was fed into the upper end of the HMS model was based on an unregulated flow frequency analysis plus reservoir routing simulations, it truly represents an n-year flow event. For index points downstream of Bellota, the hydrographs represent what would happen when you have an n-year event centered above Bellota and concurrent runoff downstream. For these reasons, the hydrographs produced in the HMS model are probably not significantly different than if PBI had created a specific storm centering for each and every index point: 1) the majority of runoff that gets into the levee system comes from sources above Bellota (approximately 75% or more) 2) the lower watershed is heavily leveed downstream of Bellota and only a few locations exist where water can enter into the levee channels.

2.0 HMS model calibration

2.1 Background: The firm PBI performed rainfall runoff modeling for the Lower SJQ River Feasibility Study. PBI developed an HEC-HMS model for the lower Calaveras River downstream of New Hogan Dam. This model was then integrated with a separate reservoir modeling analysis of New Hogan Dam, performed by David Ford Consulting Engineers (DFCE), in which the flow at Bellota (for the 1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, and 1/500 AEP events) would be derived from the reservoir operation analysis, and coincident local flows below the Bellota gage would be derived from the HEC-HMS model developed by PBI. Rainfall runoff model calibration was performed for the local flow areas between New Hogan Dam and the Bellota gage. Modifications to the base parameters needed to match the observed flow at Bellota for the April 2006 storm event were then applied to the ungaged watershed areas in the study. PBI calibrated their model using the recorded hourly flow at the Mormon Slough at Bellota gage. The flow at this location represents both New Hogan Dam releases and local flow runoff from the approximately 107 square mile area between the dam and the stream gage. To accomplish getting a similar hydrograph from their model, PBI took the recorded reservoir outflow hydrograph shown in figure 1 shown below and routed it from New Hogan Dam location to the Bellota index point where it was combined with the local flow hydrograph produced by their rainfall runoff simulation. For their final simulation, PBI adopted a basin “n value” of 0.15 and constant soil loss rates of 0.85 times the handbook values. The final calibration run with their adopted parameters is shown in Figure 2.

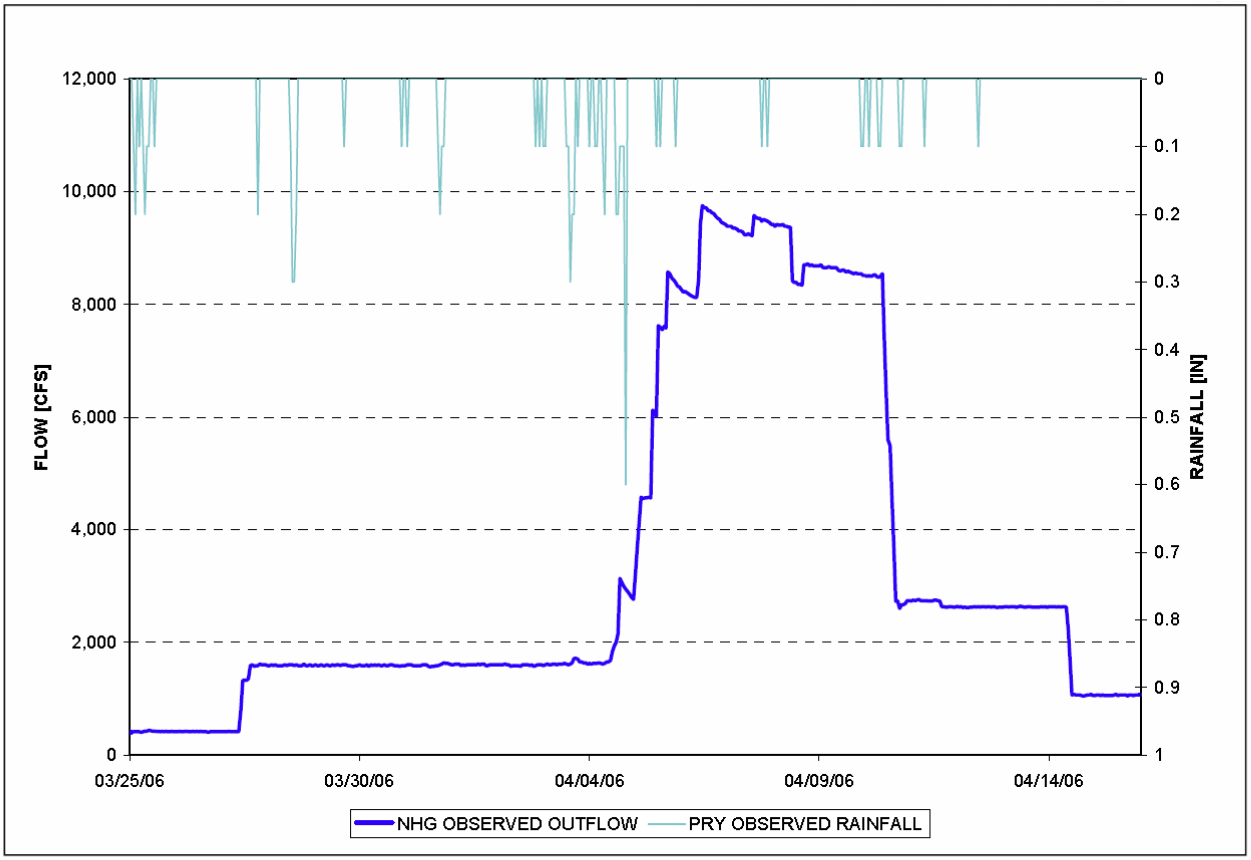


Figure 1: Observed outflow from New Hogan Dam. This hydrograph was routed downstream and added to local flow computed in HMS.

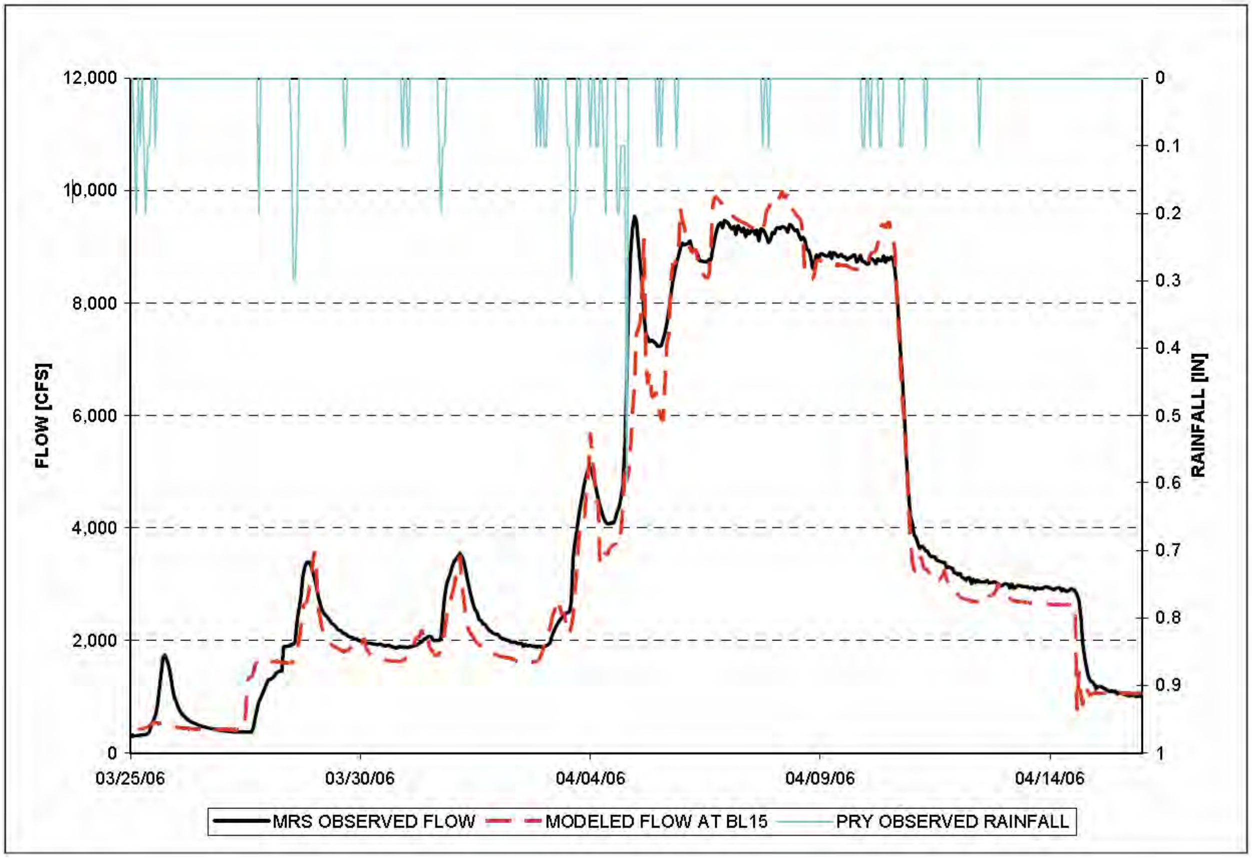


Figure 2: Observed and modeled flow at Bellota. Both hydrographs include both outflow from New Hogan Dam and additional local flow contributions.

2.2 Issue: During the ATR review of the Hydrology Appendix, it was recommended that the calibration results be compared for the local flow below New Hogan Dam only, rather than total flow at the Bellota gage (which includes New Hogan Dam outflow). DFCE had previously developed hourly local flow hydrographs by subtracting observed reservoir releases (routed downstream to Bellota) from the total flow observed at the Bellota gage. SPK provided the Ford local flow hydrographs for the 1997 and 2006 floods to PBI. PBI then performed calibration runs without the New Hogan Dam reservoir releases. Initial results for the 1997 calibration run are shown in figure 3. The model came up significantly short in peak and volume. An attempt was made to lower the basin n (shorten lag) but the timing of the peak became too early as shown in figure 4. The basin n was then restored back to 0.15. Next, several attempts were made to adjust which precipitation gages were assigned to each rainfall zone, and by dropping soil loss rates down to the lowest range possible (per handbook guidance). The model still came up short in peak and volume! The only positive result from the calibration was confirmation of an appropriate basin n value of 0.15 as it also worked well for the 2006 calibration effort. The effort to use the 1997 event for calibration was abandoned since precipitation data was apparently too low (insufficient).

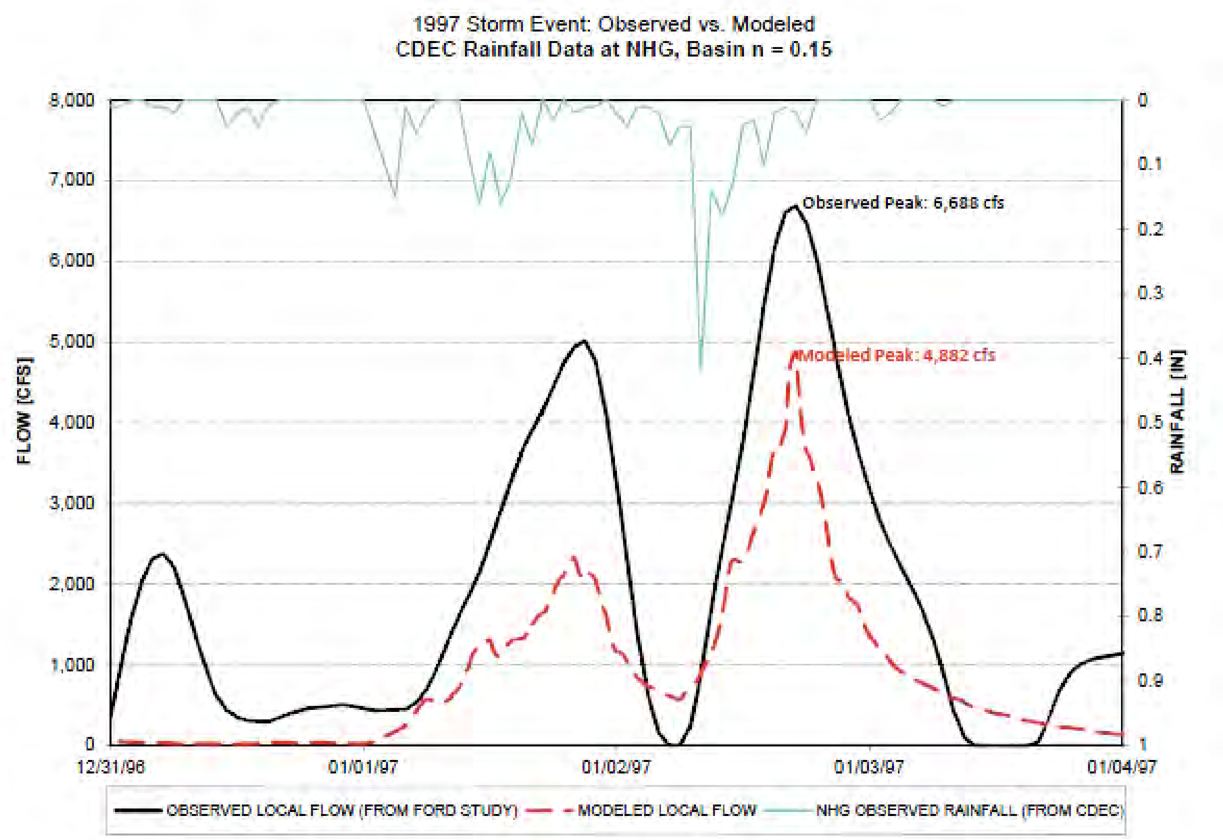


Figure 3: Initial comparison of observed (computed from gage data) to simulated local flow between New Hogan and Bellota (1997 event). Both peak and 3-day volume were found to be low.

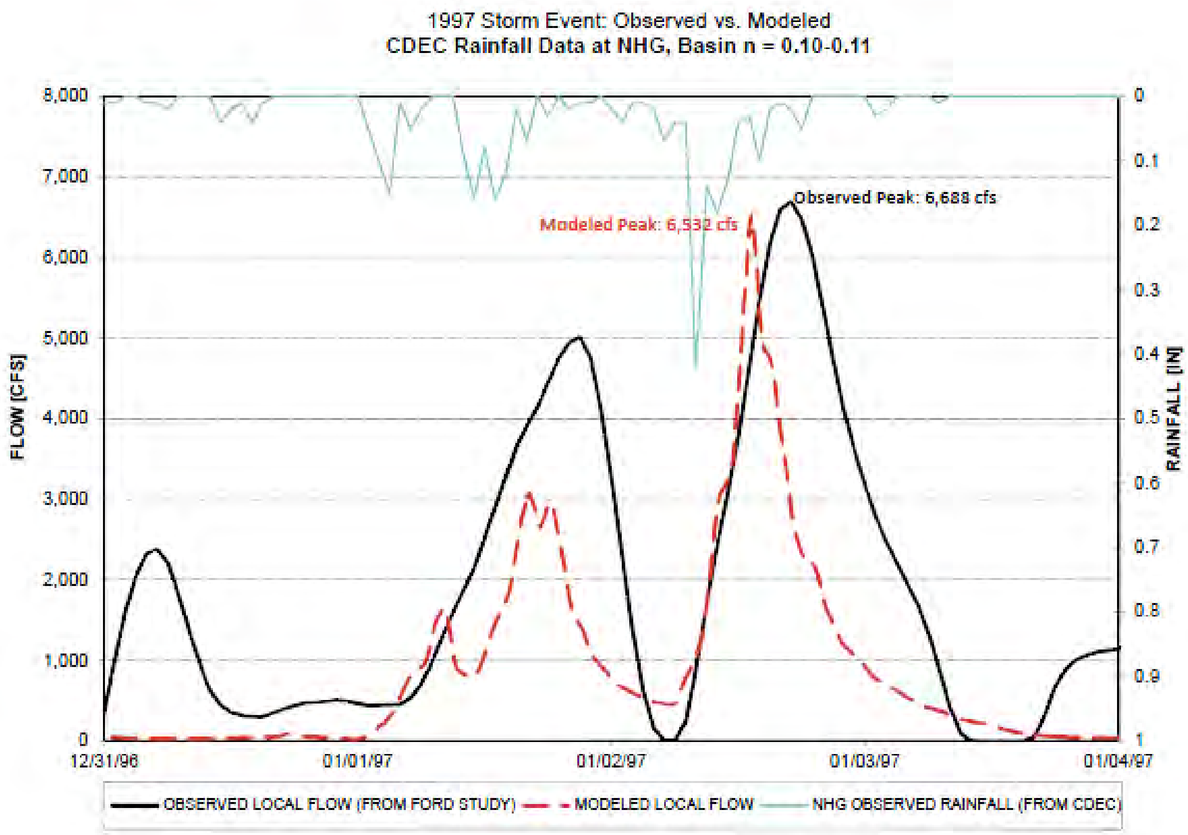


Figure 4: 1997 event calibration with reduced basin n.
Note: Timing is too early.

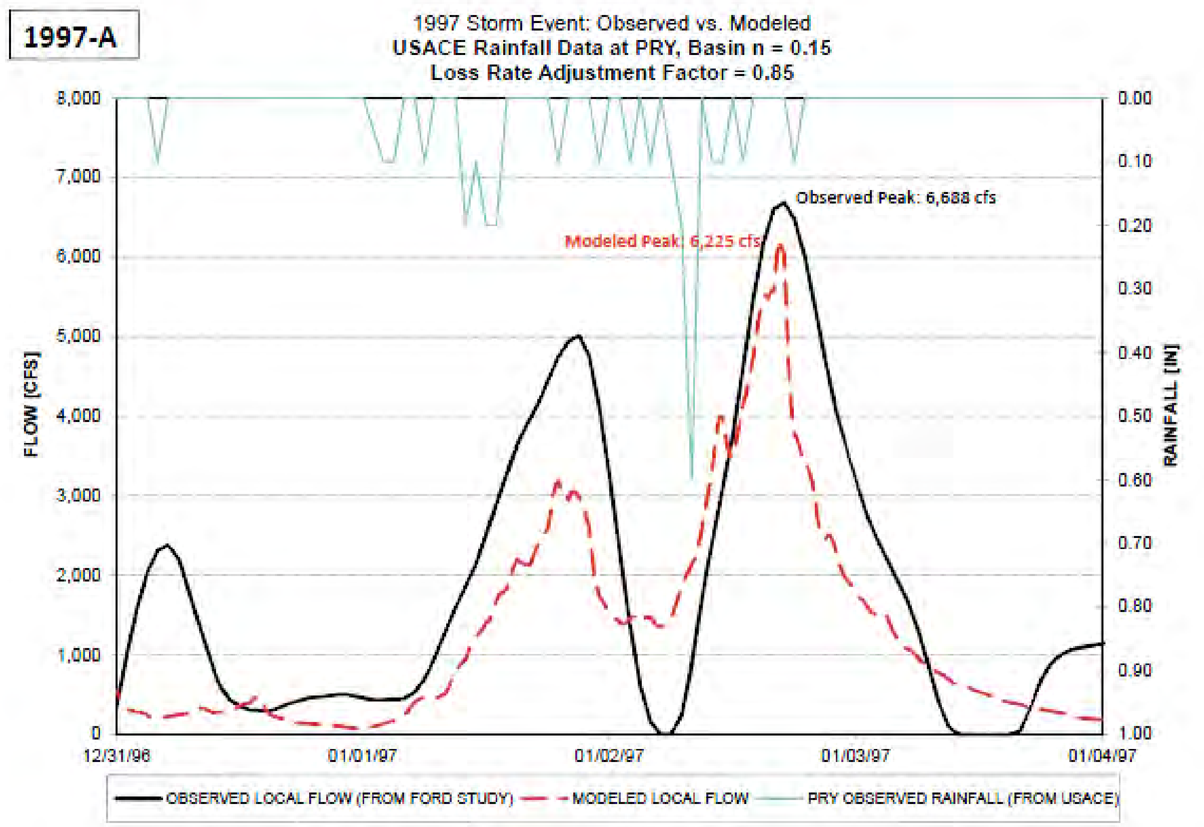


Figure 5: 1997 event calibration using adjusted rainfall ..
Note: Peak and volume still comes up short.

2.3 2006 Event Calibration. Next, the model was re-calibrated to the 2006 flood event. The intial calibration run resulted in Figure 6 below. The model came up short in peak and volume.

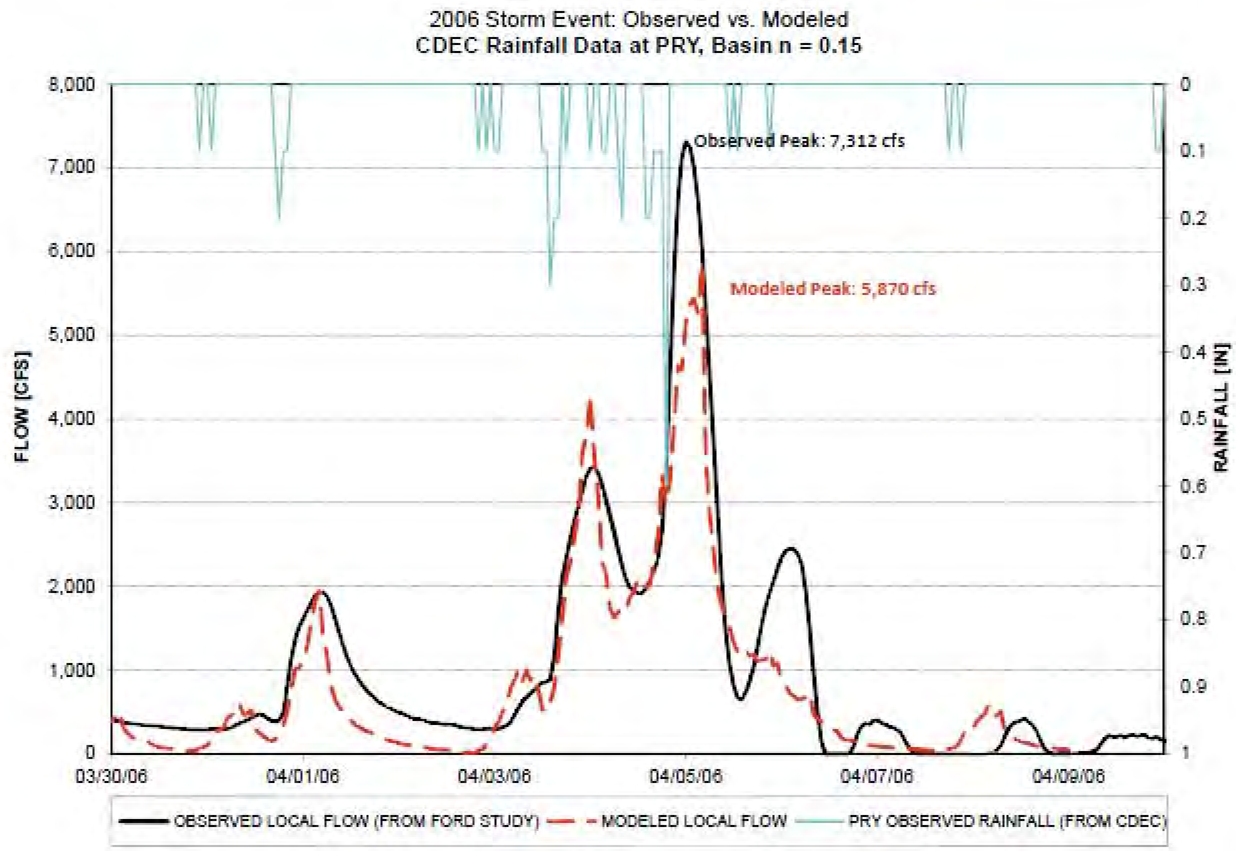


Figure 6: Initial 2006 event calibration
Initial comparison of observed (computed from gage data) to simulated local flow between New Hogan and Bellota (2006 event). Both peak and 3-day volume were found to be low.

2.4 Resolution: Several modifications to the HMS model parameters were investigated, before a final calibration was adopted:

1) The unit hydrograph parameter "basin n" was modified to create a more peaked unit hydrograph. This modification caused mixed results as shown in figure 7. The waves around either side of the main wave appear to occur too early as compared to figure 6. As this did not seem desireable, the original basin n value of 0.15 was restored.

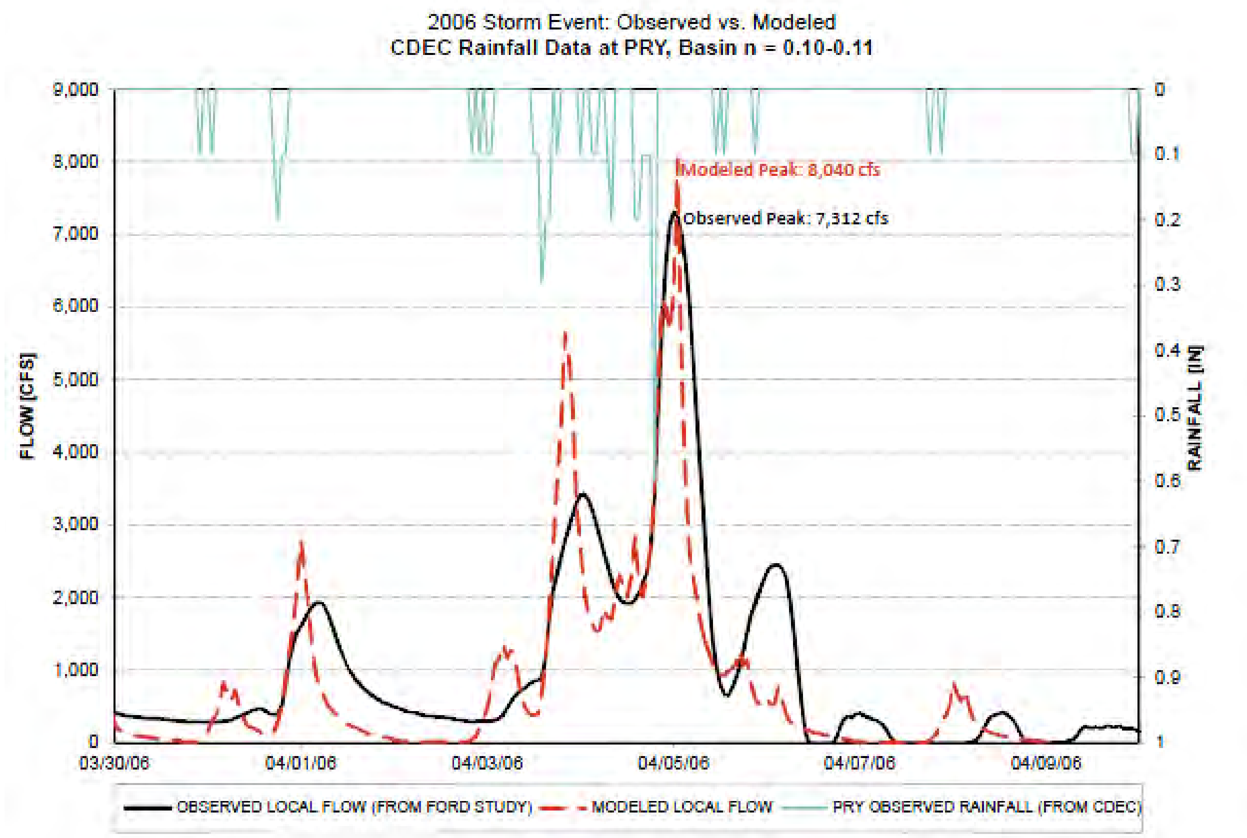


Figure 7: 2006 Event with reduced basin n.
Note: Attempts to match peak by reducing the basin n were found to provide mixed results; peak of the main wave is okay but the pre- and post-waves happen earlier than figure 6

2) The next attempt to get a better match was to lower soil loss rates. In order to get a good match to peak and volume, some soil types had to be lowered below the lower limit of the range suggested in handbooks. Consequently, this adjustment was abandoned and HEC-HMS soil loss rates adopted by PBI were restored (85% times the average soil loss rate per soil type).

3) The last step was to modify the precipitation. To do this, logically based re-assignments were made as to which observed gage hyetographs were assigned to each subbasin, on the basis of comparative proximity and representative elevation. For the 2006 calibration, the model performed very well with this adjustment. The gages used in the calibration are shown in Figure 8.

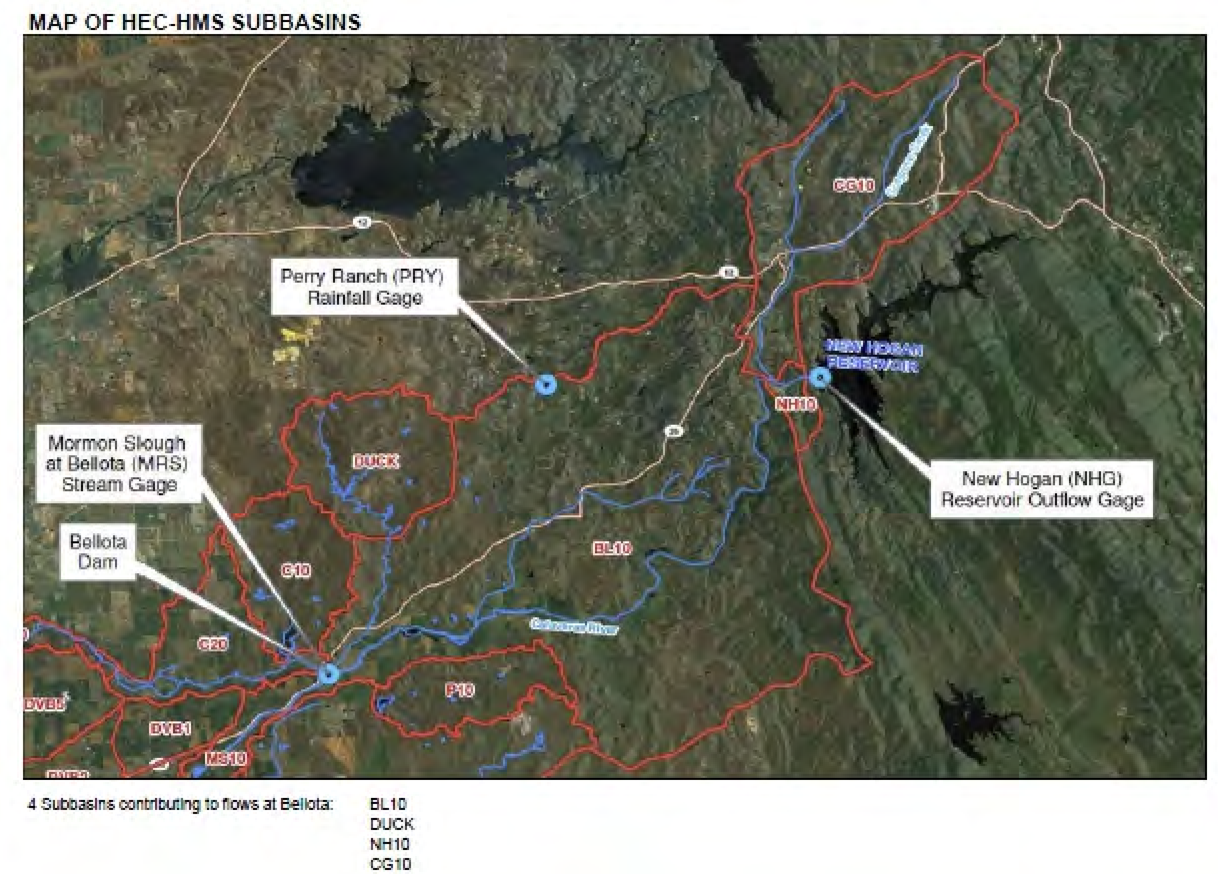


Figure 8: Precipitation gage locations in relation to the subbasins comprising the local flow between New Hogan Dam and the Mormon Slough at Bellota gage. Rainfall at the New Hogan gage is believed to be more representative of CG10 and NH10 subbasins, while an average of both Perry Ranch and New Hogan seems appropriate for the local areas downstream. Note: this applies to the 2006 event. Precipitation recorded at New Hogan appears to be significantly underreporting in relation to the volume of runoff observed.

4) The final calibration run for 2006 used the original calibration parameters of a basin n of 0.15 and average constant soil loss rates from the handbook times 0.85. The rainfall was modified per discussion under Figure 8. The final adopted calibration run is in Figure 9 below.

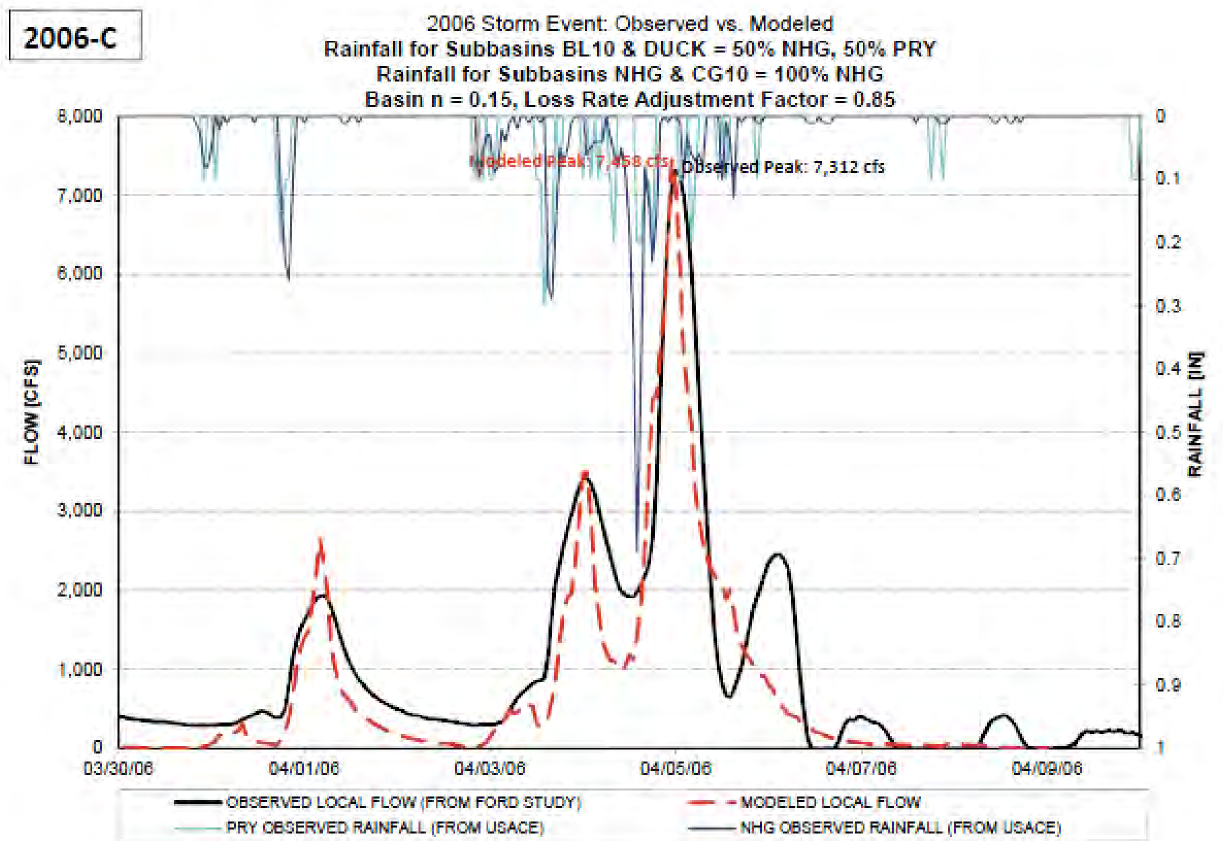


Figure 9: Calibration results for the 2006 event were significantly improved with the modified precipitation gage selection.
Note: The original PBI calibration parameters of basin n = 0.15 and constant soil loss rates = 85% of average handbook values was used for the final calibration run.

2.5 Summary: A reasonable recreation of the 2006 event was achieved by reassigning the observed precipitation gages used for each subbasin to those logically expected to be representative of their respective drainage area. For the 1997 event, no matter which gages were assigned to each subbasin, the model always came up short in peak and volume. Ultimately, this calibration was abandoned as it was realized that the rainfall data was not adequate to accurately model this event. In conclusion, the new calibration efforts reinforced to the Corps that the original model parameters that were adopted by PBI for the n-year simulations were acceptable. As such, no adjustment of the hydrology was deemed necessary.

3.0 Design Storm Sensitivity Analysis

Background: The original design storm created by PBI used a 1997 pattern storm that was balanced to the average 72-hour depth found in NOAA Atlas 14 for the 140 square mile area downstream of the Mormon Slough at Bellota gage. This was justified by a test that the firm performed earlier in the study. In this test, PBI used the balanced storm feature in HEC-HMS (balanced to the 1-hour through 72-hour NOAA14 depths) for a 0.005 AEP storm centered over the area between New Hogan Dam and the Bellota gage. HMS automatically applies TP 40 areal reduction factors to this storm. This was compared to HEC-HMS results when an observed 1997 hyetograph pattern was balanced only to the NOAA14 72-hour depth for the 0.005 AEP event (with HMR 59 areal reduction applied for a 72-hour duration). The resulting peak flow was 12,500 cfs in both cases.

Issue: During the ATR review of the Hydrology Appendix, concern was expressed about the design storm including: a) balancing the pattern hyetograph to only the 72-hour duration b) applying areal reduction only to the 72-hour depth of the design storm rather multiple durations and c) PBI's use of an average of two types of centerings to determine the 72-hour areal reduction factor. These two centerings were the "above New Hogan Dam centering" and the Bellota Centering (storm centered on the area between the dam and the Bellota gage.

Resolution: To address the above concerns, the Corps created a new 0.01 AEP balanced design storm to run in the model for comparison with the PBI design storm results. The Corps' design storm used a 1997 pattern hyetograph that was manipulated/balanced to the 1-hour through 72-hour NOAA14 depths (1-, 3-, 6-, 12-, 24-, 48-, and 72-hour durations).

The areal reduction factors applied to this new design storm were designed to produce concurrent rainfall downstream of Bellota when the entire drainage area upstream of Bellota was having a 0.01 AEP storm (storm that creates 0.01 AEP runoff at the Bellota gage). The following steps were utilized to determine the depths to use in each subbasin:

1. For each duration (i.e 1-hour, 2-hour, etc), use GIS to determine the average 0.01 AEP NOAA14 point rainfall for the entire watershed upstream of the Bellota gage.
2. Apply the appropriate HMR 59 areal reduction factors to the point precipitation depths found in step 1 (use reduction factor for drainage area above the Bellota gage) .
3. Multiply the areally reduced depths found in step 2 by the drainage area upstream of the Bellota gage to get a volume of precipitation (per duration).
4. Repeat steps 1 through 3 for the entire watershed of the Calaveras River. Compute the rainfall volumes (per duration) for the entire watershed.
5. Subtract volume found in step 3 from the volume found in step 4 for the entire watershed. This must be done for each duration (i.e. 1-hour, 2-hour, 3-hour, etc). The result is the remaining volume that can be applied to the watershed area downstream of

Bellota (i.e. when the whole watershed is incurring a 0.01 AEP event with a specific "bullseye" above Bellota).

6. To account for orographic influences (rather than apply the same depth to all subbasins), find the relative "weighting" of every subbasin that is downstream of Bellota. First, multiply each subbasin area by its mean annual precipitation (MAP). Each subbasin's MAP can be found using GIS. The multiplication will create a volume "x". Next, divide each subbasin's "x" by volume "y" (total area ds of Bellota times its MAP). This will result in a ratio/percentage which is the percentage of volume found in step 5 that is to be applied to each subbasin.
7. Finally, divide the volume allotted to each subbasin (based on step 6) by the subbasin drainage area. This is the depth (per duration) that is to be applied to the design storm for each subbasin.

Summary: The above design storm was run in the HEC-HMS model for two scenarios. One scenario included applying the Bellota hydrograph (which includes New Hogan Dam releases) at the upstream end of the model. The other scenario only looked at the differences in local runoff created by the HMS model (without the Bellota hydrograph). The table below provides a comparison of results between the PBI design storm versus the Corps revised design storm at the farthest downstream end of the Calaveras River. This comparison demonstrates the hydrographs in the HMS model are reasonable and do not need modification.

Below is a comparison summary of results at the model outlet:

	Local Flows Only		With New Hogan Outflows	
	Peak Flow [cfs]	Total Volume [AF]	Peak Flow [cfs]	Total Volume [AF]
Current LSJRFS Storm	3,208	7,947	15,603	247,331
Fully Balanced Storm	3,150	7,660	15,544	247,125
% Difference	-1.8%	-3.6%	-0.4%	-0.08%

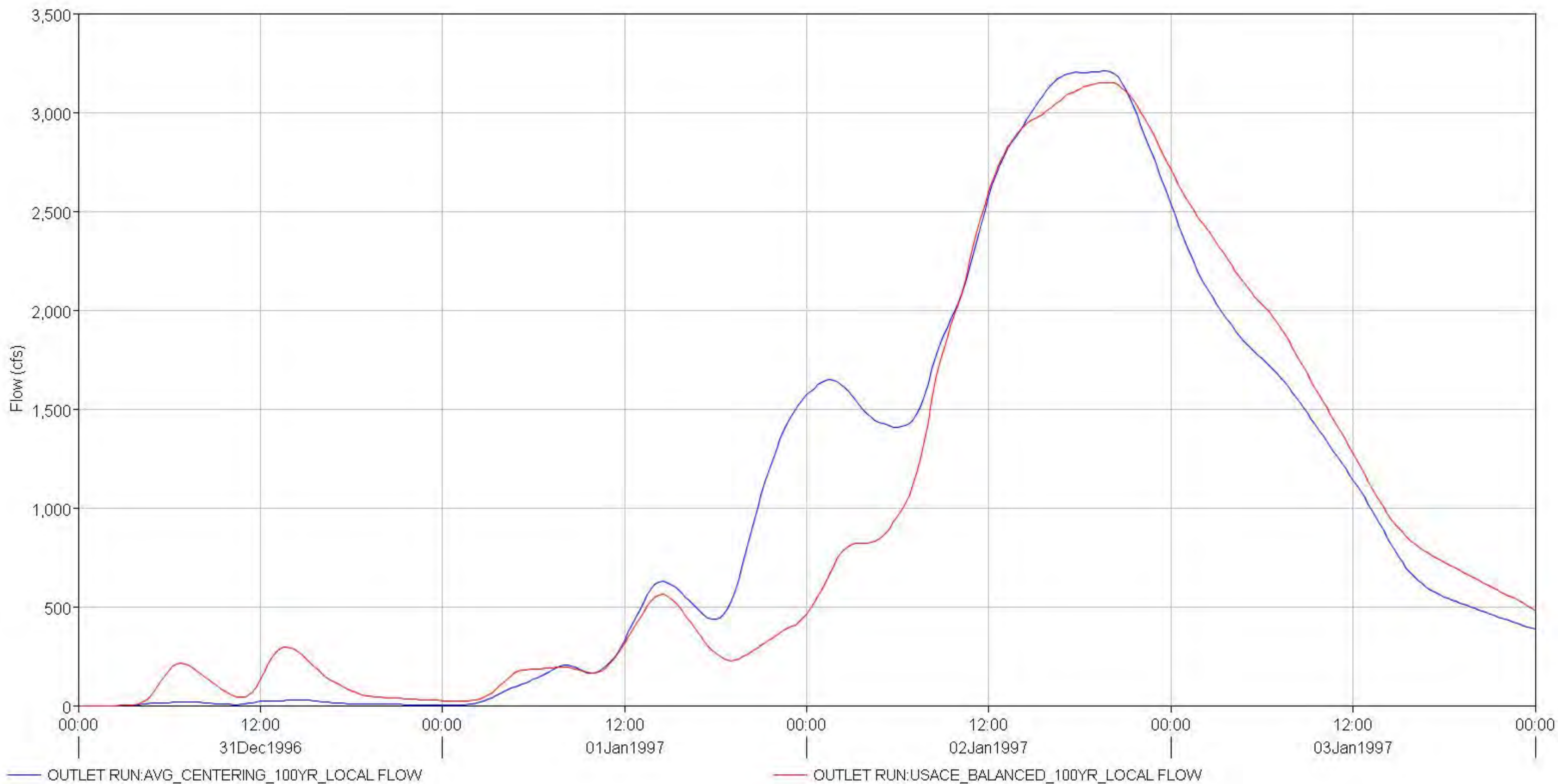


Figure 4: Comparison of local flows only. Blue: original study results using design storm scaled to 3-day duration and area reduction factors. Red: Results using a fully balanced design storm and areal reduction factors.

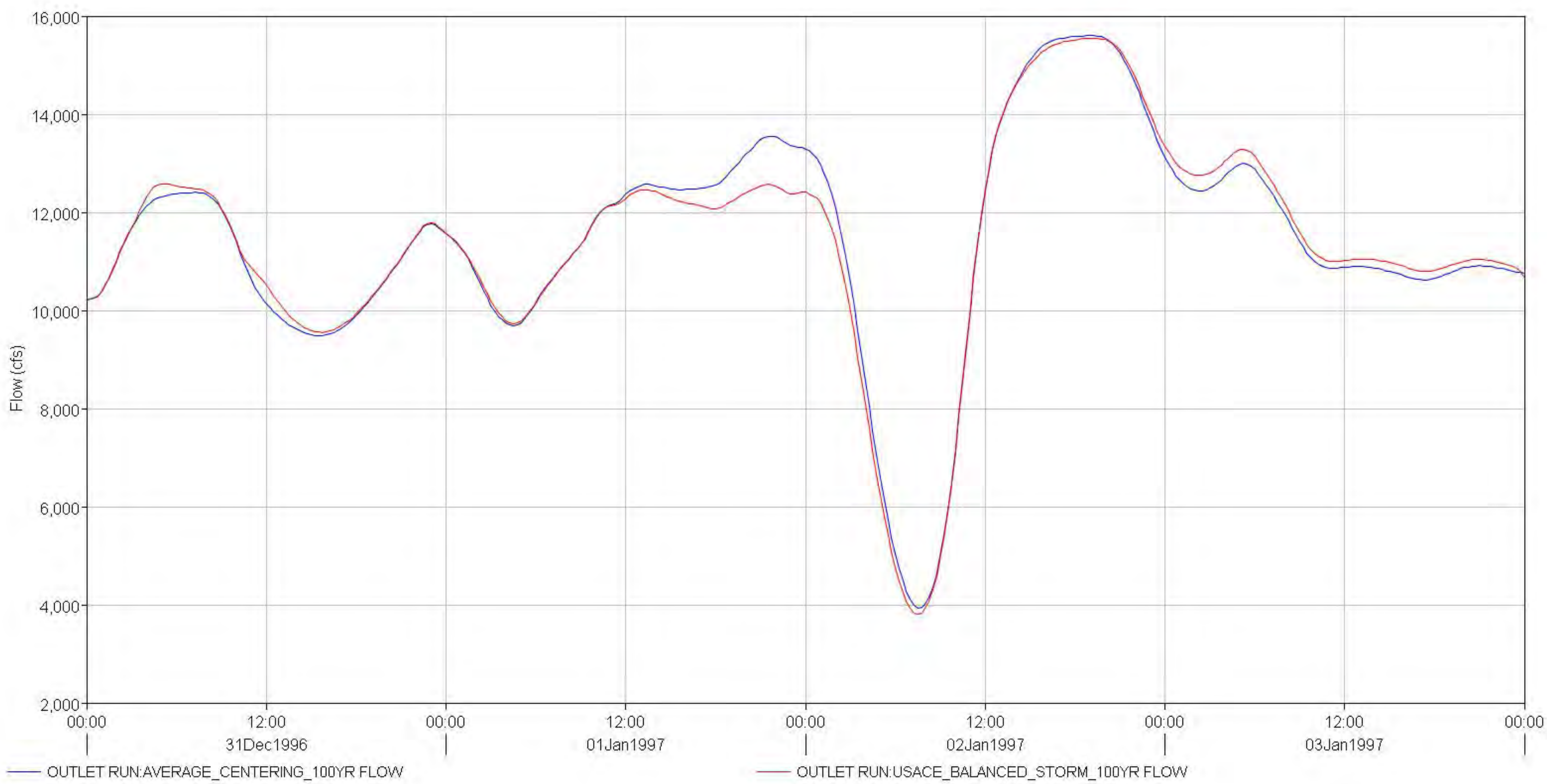


Figure 5: Comparison of total flow at model outlet. Blue: original study results using design storm scaled to 72-hour depth and area reduction factor. Red: Results using a fully balanced design storm and areal reduction factors.

3.1 Applicability to Littlejohn Creek Design Storm

The above sensitivity analyses comparing the results of a fully balanced design storm on the lower Calaveras River to the PBI design storm for the same area may indicate that the HMS modeling results for the Littlejohn Creek below Farmington, CA are reasonable. Like the Calaveras River design storm, PBI used an average of two centerings to create the design storm that was applied to the HMS model areas downstream of Farmington, Ca. These two centerings were the "upper watershed" centering (stress the foothill region) and the "Farmington" centering which stressed the watershed above Farmington Dam. The drainage area downstream of the Calaveras River at Bellota gage is 140 square miles while the drainage area downstream of Littlejohn Creek at Farmington, Ca is 182 square miles. Furthermore, the flow hydrograph on the lower Littlejohn Creek is bifurcated four times and highly attenuated in storage areas downstream of Farmington, Ca which makes the local flow below the Farmington gage less important for this watershed. At the confluence of Littlejohn Creek and Duck Creek where the French Camp Slough levees begin, specific frequency events centered on the mainstem San Joaquin River cause the highest stages due to backwater. The specific frequency flows coming down the tributary do not cause the highest stages within the French Camp Slough levees. To date, the feasibility study has not found an alternative for Littlejohn Creek due to a lack of sufficient annualized damages to justify a project. As the unregulated flow frequency curves at Farmington, Ca are probably conservative (flows on the high side) as stated in Appendix 2, the hydrology has not negatively impacted the study goals.

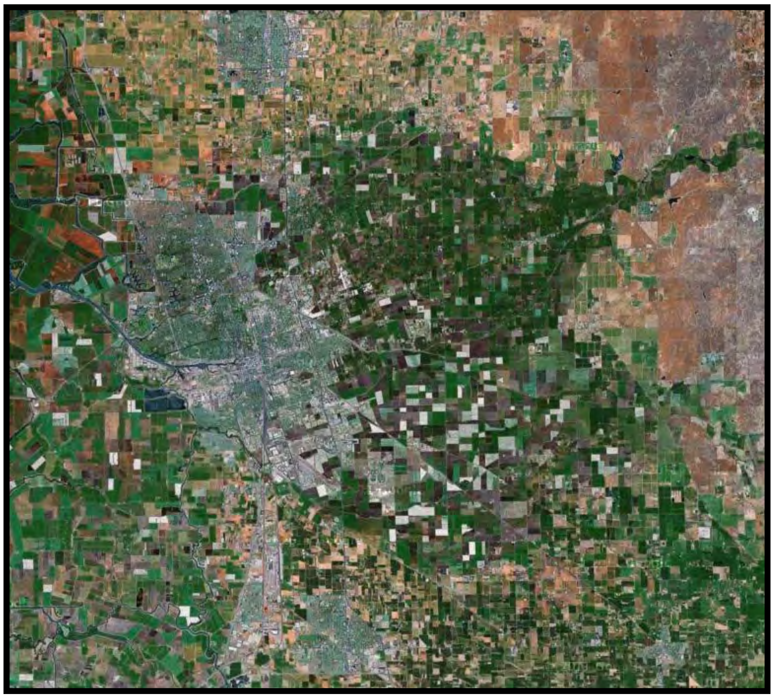
3.2 Bear Creek Design Storm

A 1997 pattern hyetograph fully balanced to multiple duration NOAA14 precipitation frequency depths was used for the study which meets USACE guidelines. The PDT team used the storm centering that caused the worst flow on the Bear Creek for its analysis (assess floodplain damages). From a statistical viewpoint, SPK agrees that an average centering is the more desireable method to provide a best estimate of a specific frequency flow at an index point. Regardless, the feasibility study found that annualized damages on Bear Creek were not high enough to justify a project. As the hydrographs used for the floodplain analysis were probably conservative (too high), the hydrology did not negatively impact the study goals.

Attachment 1

Lower San Joaquin River Feasibility Study, F3 Hydrology Appendix by Peterson, Brustad, Inc. dated July 30, 2012.

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY



F3 HYDROLOGY APPENDIX

JULY 30, 2012

US Army Corps of Engineers

SJAFCA San Joaquin Area FLOOD CONTROL Agency

DEPARTMENT OF WATER RESOURCES STATE OF CALIFORNIA

PREPARED BY:

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ENGINEER'S SIGNATURE PAGE

This report titled:

Lower San Joaquin River Feasibility Study:
F3 Hydrology Appendix

has been prepared by or under the direct supervision of the following registered Civil Engineers:

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1.0 INTRODUCTION

U.S. Army Corps of Engineers (USACE) in conjunction with the San Joaquin Area Flood Control Agency (SJAFCA) and the Central Valley Flood Protection Board is preparing the Lower San Joaquin River Feasibility Study (LSJRFS) to evaluate flood damage reduction projects within the Lathrop to Stockton urban and urbanizing corridor, which includes Bear Creek, Mosher Slough, Calaveras River, French Camp Slough, and Lower San Joaquin River watersheds. This section of the F3 Hydrology Report documents the HEC-HMS model preparation and resulting flows within the study watersheds, with the exception of the Lower San Joaquin River, which will be documented by USACE under separate cover. Hydrologic modeling was performed with a 72-hour storm for the 1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, and 1/500 annual exceedance probability (AEP) events.

2.0 DESIGN STORMS

Design storms with 72-hour durations were created for the 1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, and 1/500 AEP events as input to the LSJRFS HEC-HMS models. As discussed in Section 2.3, the 72-hour storm pattern provides a storm event that is high in both peak flow and volume which is important for levee breach scenarios.

2.1. RAINFALL ZONES

LSJRFS subbasins were aggregated into 7 rainfall zones with uniform rainfall characteristics. Seven rainfall gages were selected to form the basis of this subbasin aggregation. The selected gages are distributed throughout the study area and have available rainfall data at short-interval timesteps which can be used for storm patterning (see Section 2.3).

GIS software was used to draw Thiessen polygons around the selected rainfall gages and subbasins lying within each Thiessen polygon were aggregated to create the rainfall zones (Figure 2- 1).

2.2. DESIGN STORM DEPTHS

The National Oceanic and Atmospheric Administration (NOAA) published its *Atlas 14 Precipitation Frequency Study for California*[1] in April 2011 which includes estimates for design rainfall depths in an ASCII grid file format for use in GIS. A shapefile with 7 defined rainfall zone boundaries was projected on top of the NOAA14 ASCII grid files to calculate average point rainfall depths within each rainfall zone for 96 different frequency-duration combinations.

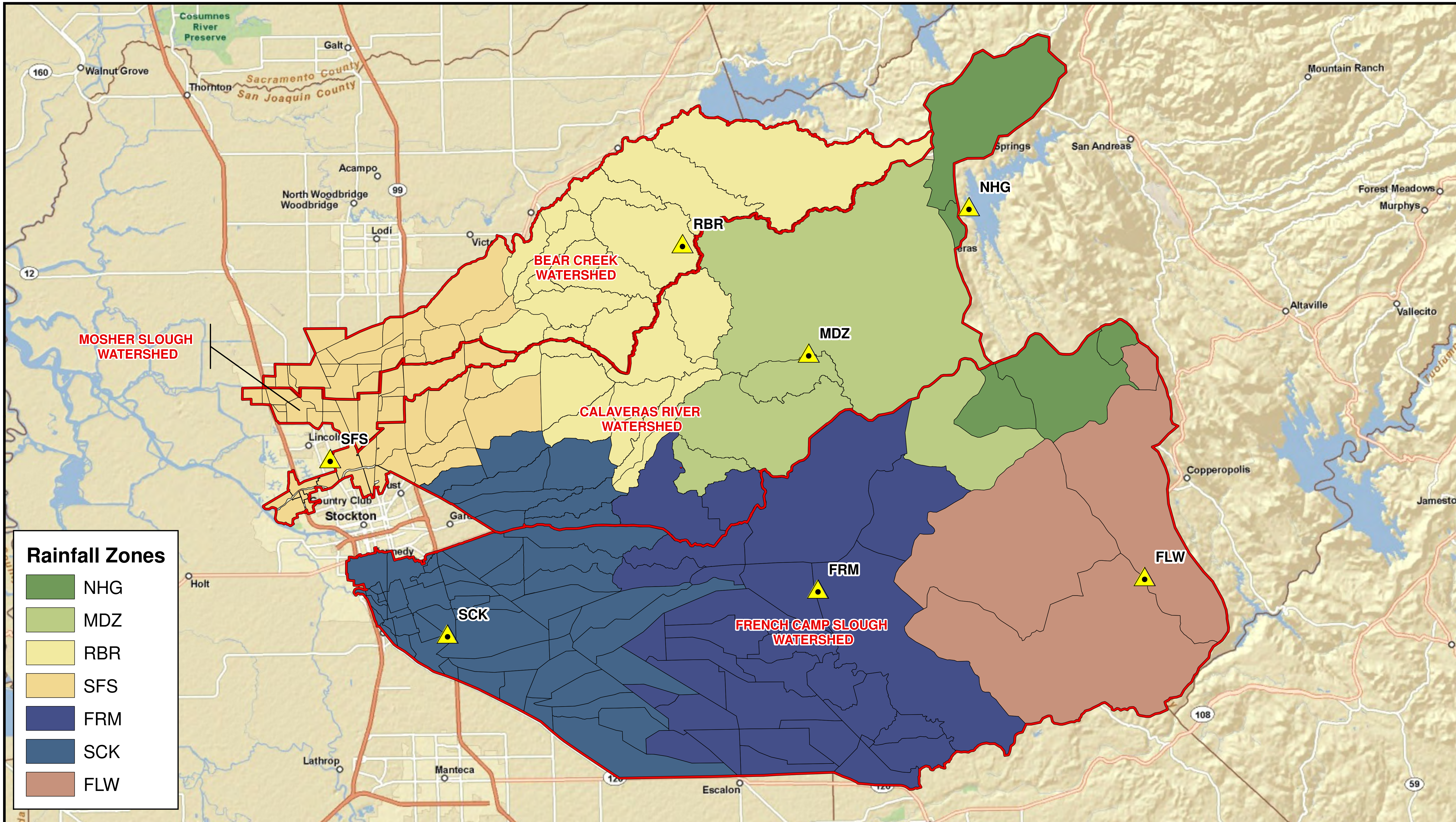
The output from the NOAA14 GIS data acquisition process includes depth-duration-frequency tables for each rainfall zone. These depth-duration-frequency tables are included for each watershed in their respective attachments.

2.3. DESIGN STORM PATTERN

The design storm pattern used for the LSJRFS is based on an observed storm event that was recorded at various rainfall gages within the study area.

The December 31, 1996-January 3, 1997 rainfall event (1997 Event) and the April 2, 2006-April 5, 2006 rainfall event (2006 Event) were considered for the basis of design storm patterning. These events represent two of the largest storms in recent history.

Data records were checked for these events at all known precipitation gages within the vicinity of the study area. Some gages only had recorded data at monthly or daily intervals and were excluded from the gage selection process based on their inadequate time step. Other gages were excluded due to lack of data for the specific dates listed; many of the available rainfall gages did not contain data for the 2006 Event.



Watershed Boundary

Precipitation Gage

Subbasin Boundary

0 5 Miles

1 : 250,000

APRIL 25, 2011

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LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

LSJRFS Rainfall Zones

FIGURE

2-1

The 1997 Event is often considered an industry standard for rainfall events and was ultimately selected as the pattern used to temporally distribute the design storms.

Data from the New Hogan (NHG) gage location represents a typical 72-hour hyetograph pattern for the 1997 Event and is shown below.

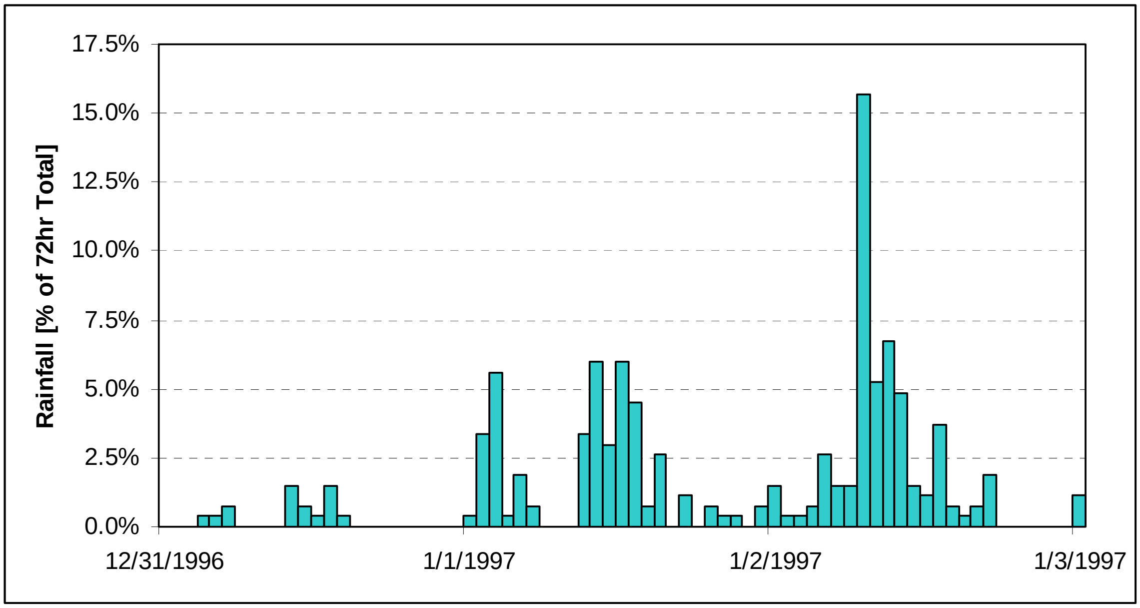

Figure 2- 2. Typical Rainfall Pattern for the 1997 Event.

The 72-hour storm pattern provides a storm event that is high in volume which is important for levee breach scenarios. For the LSJRFS, it is also desirable to preserve the high peak flows that would result from a standard, 24-hour design storm. Therefore, additional analyses were conducted to run a SCS Type 1 storm, an industry standard 24-hour storm, with the same rainfall depths to confirm that the peak flows resulting from using the 72-hour, 1997 Event hyetograph pattern are comparable to the standard, 24-hr peak flows.

All flows were comparable except for those in the Bear Creek watershed. To correct this, Bear Creek hyetographs were patterned after the 72-hour, 1997 Event and then balanced to the 3-, 6-, 12-, 24-, 48-, and 72-hour NOAA14 storm depths. After balancing the hyetographs, Bear Creek models produced high-volume hydrographs with peak flows that are comparable to those resulting from a standard 24-hour design storm.

2.4. STORM CENTERING APPROACH

The LSJRFS utilizes a storm centering approach to consider depth area reduction of design storms falling over the study area. This area reduction is typically disregarded for small watersheds where one point precipitation depth can be applied to the entire tributary area, however given the size of the watersheds in the LSJRFS it is necessary to apply area reduction factors to the point rainfall design storm depths.

Area reduction factors were calculated using a procedure that was developed by the USACE Sacramento District for the hydrology of their *Downtown Guadalupe River Project* in November 2009[2]. This procedure takes into account various storm centerings by ranking the rainfall zones according to their distance from the storm centering location and determining the cumulative drainage area for each location in the watershed. Additional details on the calculation of area reduction factors are discussed in the USACE *Guadalupe River* report provided in Attachment 2-A.

All calculated area reduction factors are included in the depth-duration-frequency tables for each watershed which are provided as attachments.

3.0 BEAR CREEK HEC-HMS MODELING

3.1. GENERAL

3.1.1. Location

Bear Creek is located near the city of Stockton in San Joaquin County, California (Figure 3-1). The watershed runs east from the city of Stockton into the Sierra Nevada foothills in Calaveras County and includes a total area of approximately 115 square miles. The uppermost portion of the watershed achieves maximum elevations of 1,000 feet and is not subject to snowmelt. It then descends through moderate slopes to the lower portion of the watershed at sea-level. The HEC-HMS model described in this memorandum has an outlet on Bear Creek at Disappointment Slough and includes Bear Creek, Upper Mosher Creek, Paddy Creek and Pixley Slough.

3.1.2. Topography

The HEC-HMS model utilized for this study is titled the PBI Bear Creek Model (PBI Model) which is georeferenced to the NAD 1983 State Plane California Coordinate System Zone III (U.S. Survey Feet). Vertical elevations are reported in the NAVD 1988 datum. Topography used for model development included United States Geological Survey (USGS) 30-meter Digital Elevation Models (DEMs)[3]. Department of Water Resources (DWR) LiDAR data[4] was also used to confirm subbasin boundaries in the lower portion of the watershed.

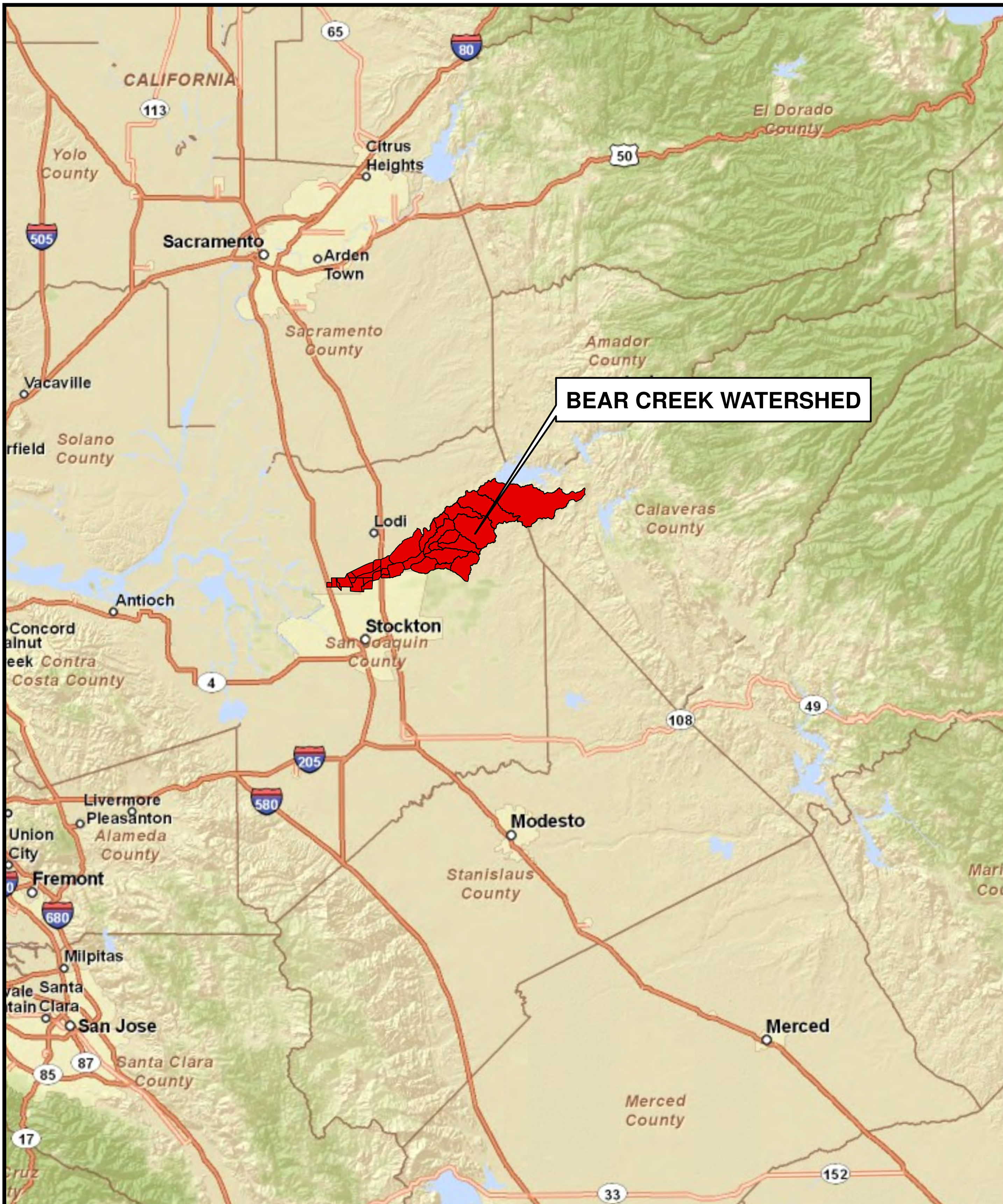

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FIGURE 3-1

VICINITY MAP OF THE BEAR CREEK STUDY AREA

3.2. MODEL DEVELOPMENT

The PBI model was developed using HEC-HMS version 3.4[5] and HEC-GeoHMS version 4.2[6]. A summary of the tasks performed are listed below:

1. The 1998 SJAFCA HEC-1 model was imported into HEC-HMS (See Section 3.2.2).
2. Subbasin boundaries were updated using HEC-GeoHMS and United States Geological Survey (USGS) Digital Elevation Models (DEMs)[1] (See Section 3.3.1).
3. Pump stations were coded into the PBI model based on design pumping rates provided by the City of Stockton[7] (See Section 3.3.2).
4. Diversions and channel routing parameters were coded into the PBI Model (See Sections 3.3.3 and 3.3.5, respectively).
5. S-graphs and lag times were assigned to each subbasin (See Section 3.3.4).
6. Loss rates and impervious percentages were coded into the PBI Model (See Section 3.3.6 and Section 3.3.7).
7. The 1/100 AEP event hyetographs from the 1998 SJAFCA HEC-1 Model were coded into the PBI Model for debugging purposes (See Section 3.2.2).
8. The PBI Model was set up to simulate both ‘Existing’ (see Section 3.5.1) and ‘Future-Without-Project’ (see Section 3.5.2) scenario runs.

3.2.1. SJAFCA HEC-1 Model

The PBI Model is a conversion and update of the HEC-1 model developed for SJAFCA by HDR Engineering, Inc. in 1998[8].

The 1998 SJAFCA HEC-1 model utilized the San Joaquin County LA preprocessor to convert S-graphs to unit hydrographs for each subbasin. Three types of S-graphs were obtained from the San Joaquin County Hydrology Manual and used based on the surface condition classification of the subbasin: Foothill, Valley Undeveloped, and Valley Developed. Lag times were calculated by HDR using basin ‘n’, length of subbasin flow, flow length from the centroid, and slope of the basin.

The 1998 SJAFCA HEC-1 model used the SCS curve number method to account for subbasin losses. Curve numbers typically ranged from 78 to 85 depending on soil type and cover. Attachment 3-A lists the parameters used in the 1998 SJAFCA HEC-1 model and compares them to the parameters used in the 2010 PBI Model.

The 1998 SJAFCA HEC-1 model was calibrated by adjusting basin ‘n’ values such that the 1/100 AEP rainfall event from the San Joaquin County Hydrology Manual produced the 1/100 AEP peak flood flow estimated for the Bear Creek at Lockeford gage. The frequency plot and statistics for this gage are provided in Attachment 3-B.

3.2.2. Conversion from HEC-1 to HEC-HMS

The 1998 SJAFCA HEC-1 model was successfully imported into HEC-HMS as the fundamental basis for the PBI Model.

Certain features in the HEC-1 software are not supported in HEC-HMS and therefore were not properly transferred during the import process. Pump station data and meteorological data from the SJAFCA HEC-1 model were manually coded into the PBI Model so as to conform to HEC-HMS formatting.

In addition, there are computational differences between the HEC-1 and the HEC-HMS software. One such difference involves the Muskingum-Cunge stream segment routing technique used for the PBI model. In HEC-HMS, channel properties are computed based on the physical characteristics of that channel, whereas in HEC-1 the properties are computed with formulas based on a kinematic wave assumption[5]. This causes minor differences in the flows that are transferred through the routing parameters. HEC-HMS results are preferred because of the refined computational techniques that have been implemented.

For initial PBI Model testing, user-specified hyetographs were assigned to each subbasin based on 1/100 AEP storm data defined in the 1998 SJAFCA HEC-1 model’s input files. This storm event was run for debugging purposes and results were made sure to match the SJAFCA HEC-1 model results. Subsequent to initial model testing, PBI modified/refined most model input elements as documented in the following sections.

3.3. MODEL FEATURES

The 1998 SJAFCA HEC-1 model was converted and modified for this study to form the PBI HEC-HMS Model. The PBI Model components are described in the following sections.

3.3.1. Subbasins

Subbasin boundaries used in the 1998 SJAFCA HEC-1 model were cross-checked with USGS 30-meter DEM datasets[3] and modified where appropriate. Subbasin boundaries were delineated using the ArcHydro and HEC-GeoHMS[6] extensions within the ArcGIS software package. These tools utilize geospatial data to interpret drainage patterns and delineate watershed boundaries accordingly. Subbasin outlet points were set similar to the locations utilized in the SJAFCA HEC-1 model. Where available, DWR LiDAR[4] data was used to confirm subbasin boundaries. In the lower portion of the watershed, west of Highway 99, subbasin boundaries were based on the City of Stockton's *Conceptual Storm Drain Master Plan*[11]. This portion of the watershed is developed and the boundaries from the City of Stockton take into account drainage improvements that have been made in the area. The Bear Creek subbasins included in the PBI Model are shown in Figure 3- 2.

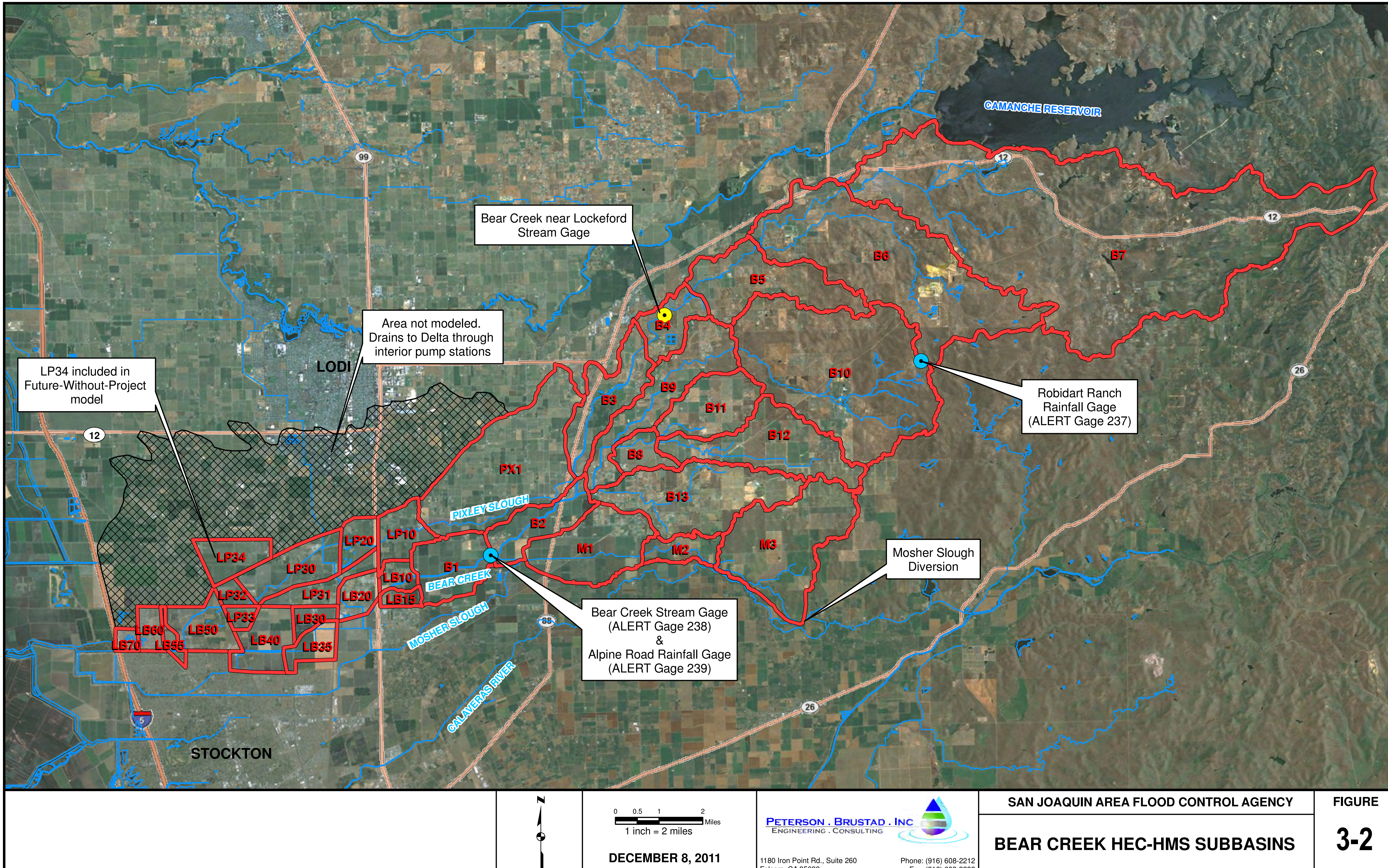
The PBI Model contains a total of 32 subbasins with drainage areas ranging from 0.26 square miles to 30.24 square miles with a total watershed area of approximately 115 square miles. An additional subbasin was added to the 'Future-Without-Project' model to account for added drainage area that is expected to be pumped into Bear Creek.

For subbasins that are on the outside of a levee which do not have pump stations, runoff is coded to enter the main channel at road crossings where there are through-levee culverts. The assumption is made that the culvert headgates will remain open and allow outside flow to enter the main channel. This assumption was made to remain conservative and to account for the potential replacement of culverts by pump stations in the future.

The GIS horizontal coordinates for each subbasin were used to georeference model elements within the PBI HEC-HMS Model. The subbasin GIS shapefile was also inserted into the PBI Model as a background map.

3.3.2. Pump Stations

Pump stations were included in the PBI Model to represent storm drain conveyance from developed subbasins to the main channels. There are three (3) pump stations included in the 'Existing Conditions' model. Multiple pumps are included at each pump station with capacities assigned based on City of Stockton records[7]. All pumps are set to discharge over the top of the levees and into the receiving channel above the highest stage expected. The exterior and interior areas at the pump stations are independent from one another.



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BEAR CREEK HEC-HMS SUBBASINS

FIGURE 3-2

Ten pump stations were then added into the 'Future-Without-Project Conditions' model to represent subbasins that are expected to become developed according to the City of Stockton 2035 General Plan[12]. Pump capacities were assigned at a rate of 0.37 cfs per acre of tributary area. This rate is based on the average flow rates of existing pump stations within the City of Stockton's systems and correlates to approximately 10-year peak flows[8]. The following table provides a summary of pump stations included in the PBI Bear Creek Model.

Table 3- 1. Summary of Bear Creek pump stations.

Pump Station	Contributing Subbasin	Subbasin Area [Sq. Mi.]	Pump Station Status	Pump Station Capacity [cfs]	Pump Station Notes
PLB6070 (I-5 PS)	LB60	0.57	Existing	46.8	3 @ 15.6 cfs
	LB70	0.26			
PLB5055 (Thornton PS)	LB50	1.54	Existing	431	Based on 0.37 cfs per acre
	LB55	0.28			
PLP33 (Pixley PS)	LP33	0.32	Existing	111	3 @ 28.1 cfs 1 @ 6.5 cfs
PLB10	LB10	0.54	Future	128	Based on 0.37 cfs per acre
PLB15	LB15	0.35	Future	83	Based on 0.37 cfs per acre
PLB20	LB20	0.83	Future	197	Based on 0.37 cfs per acre
PLB30	LB30	0.50	Future	118	Based on 0.37 cfs per acre
PLB35	LB35	0.85	Future	201	Based on 0.37 cfs per acre
PLB40	LB40	1.88	Future	445	Based on 0.37 cfs per acre
PLP34	LP34	1.25	Future	296	Based on 0.37 cfs per acre
PLP30	LP30	2.09	Future	495	Based on 0.37 cfs per acre
PLP31	LP31	1.10	Future	260	Based on 0.37 cfs per acre
PLP32	LP32	0.53	Future	126	Based on 0.37 cfs per acre

3.3.3. Diversions

All flows from Upper Mosher Creek (subbasins M1, M2, and M3), which has a combined drainage area of 9.97 square miles, are diverted to the main stem of Bear Creek at a location just upstream of the Central California Traction Railroad (see Figure 3- 2). The Calaveras River has a diversion into Upper Mosher Creek, however there are no flows going over this diversion in the winter. This diversion was originally constructed by the United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS), and improved by SJAFCA in 1998. Because the structure diverts all flow, Upper Mosher Creek was coded as a tributary area to Bear Creek. Lower Mosher Slough (downstream of the diversion), will be modeled using a separate HEC-HMS model.

3.3.4. S-graphs and Lag Times

As discussed in Section 3.2.1, the 1998 SJAFCA HEC-1 model utilized the San Joaquin County LA preprocessor for converting S-graphs to unit hydrographs. The PBI Model assigns Foothill, Valley Undeveloped, and Valley Developed S-graphs directly into HEC-HMS for each subbasin based on its location. S-graph data points were obtained from the San Joaquin County Hydrology Manual[10]. The S-graphs were developed based on rainfall-runoff data from Southern California catchments considered to be hydrologically similar to the local catchments. The following figures show the time versus discharge relationship for each S-graph.

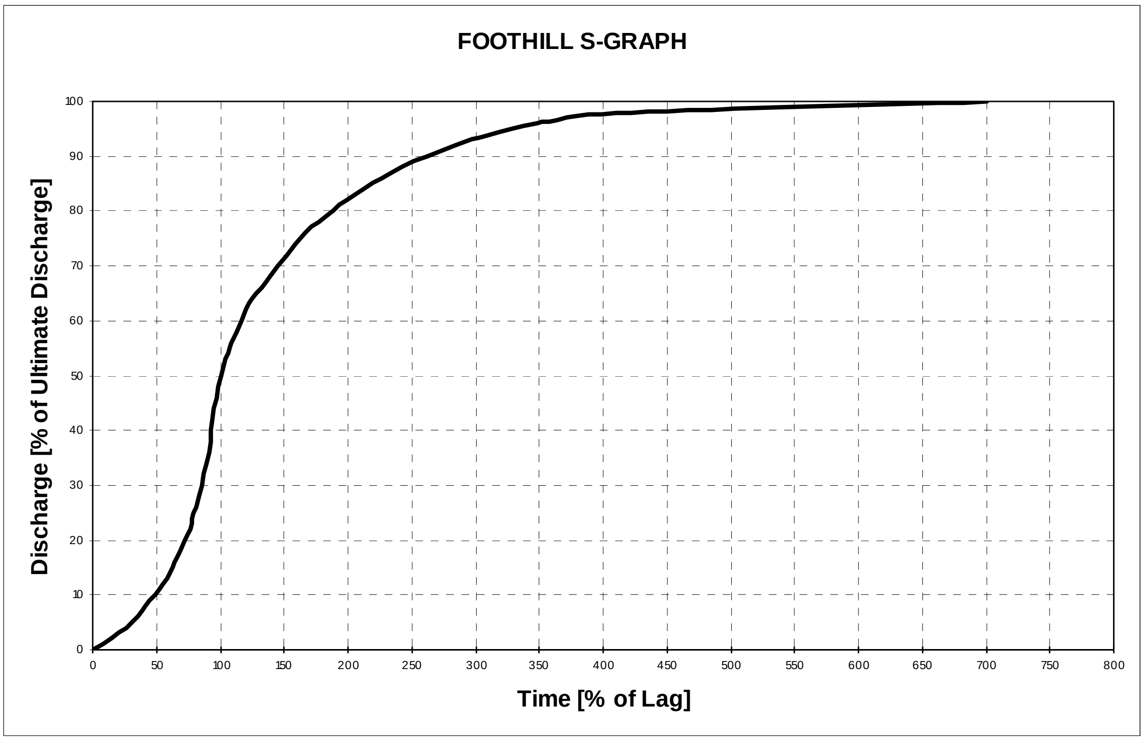


Figure 3- 3. San Joaquin County Foothill S-graph

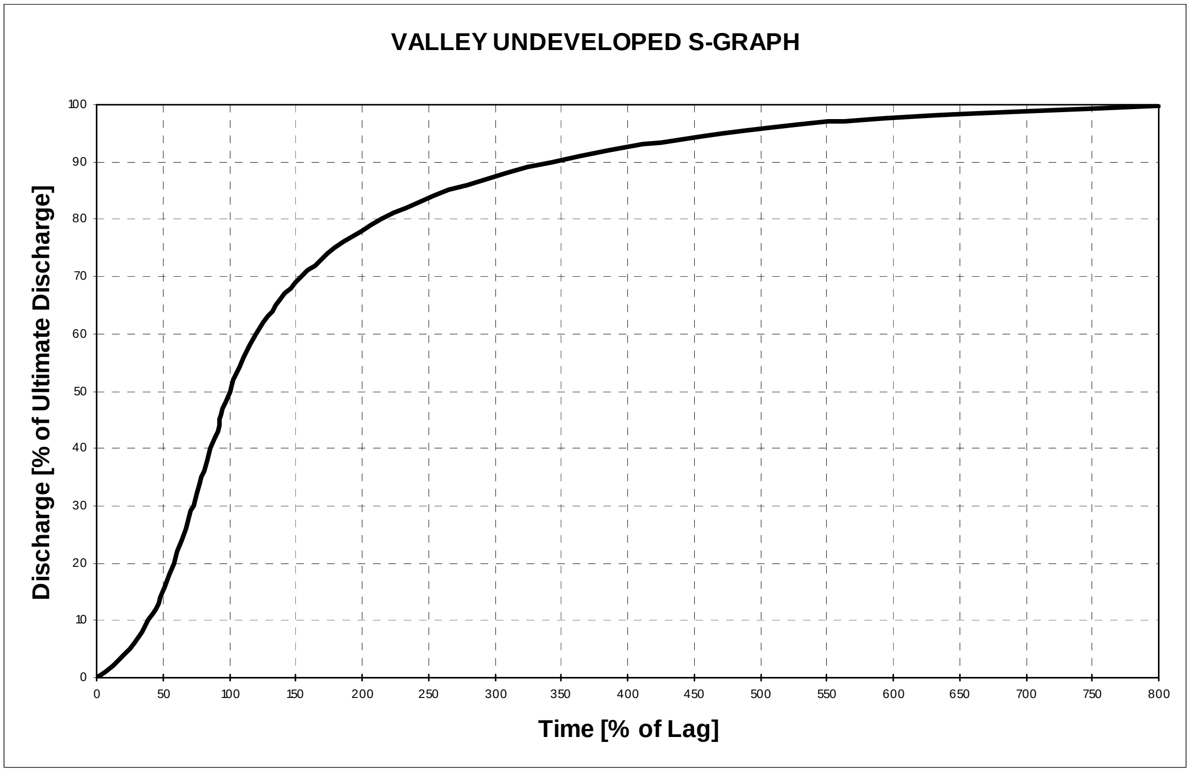


Figure 3- 4. San Joaquin County Valley Undeveloped S-graph

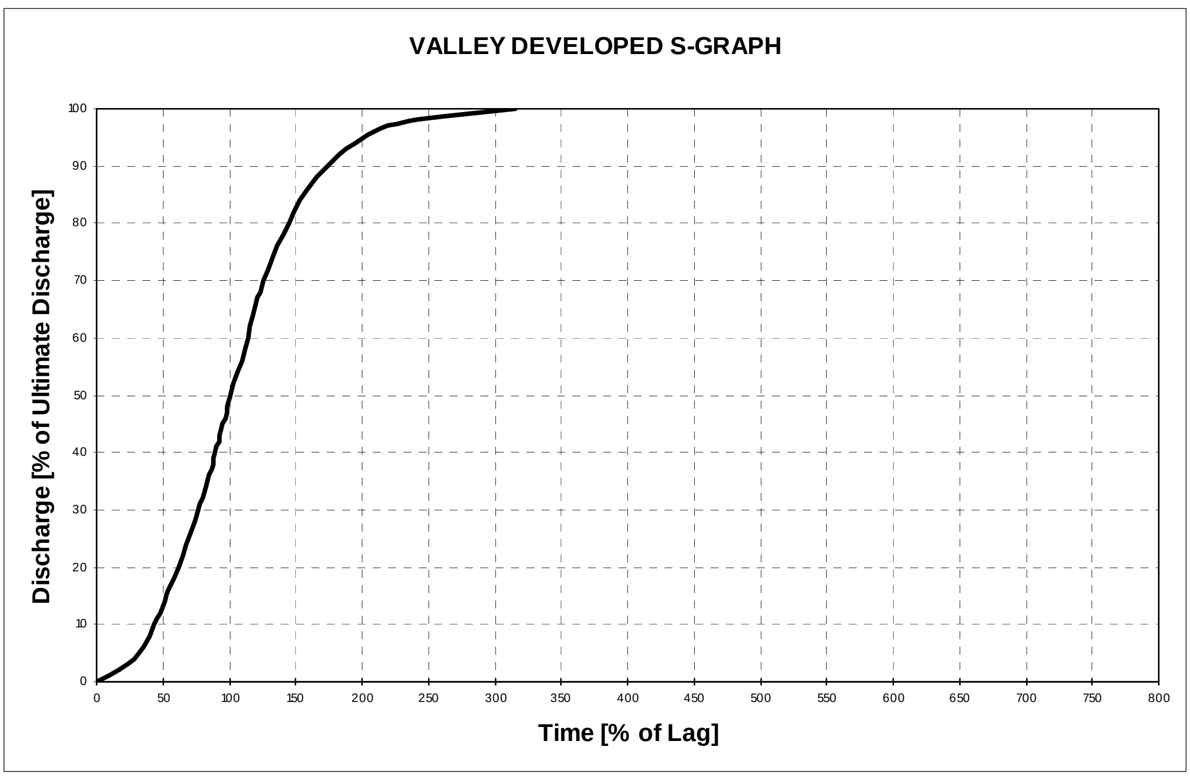


Figure 3- 5. San Joaquin County Valley Developed S-graph

Basin lag times were calculated according to guidelines set forth in the San Joaquin County Hydrology Manual[10]. The following equation was used:

$$Lg = 24n(L \cdot L_C/S^{0.50})^{0.38}$$

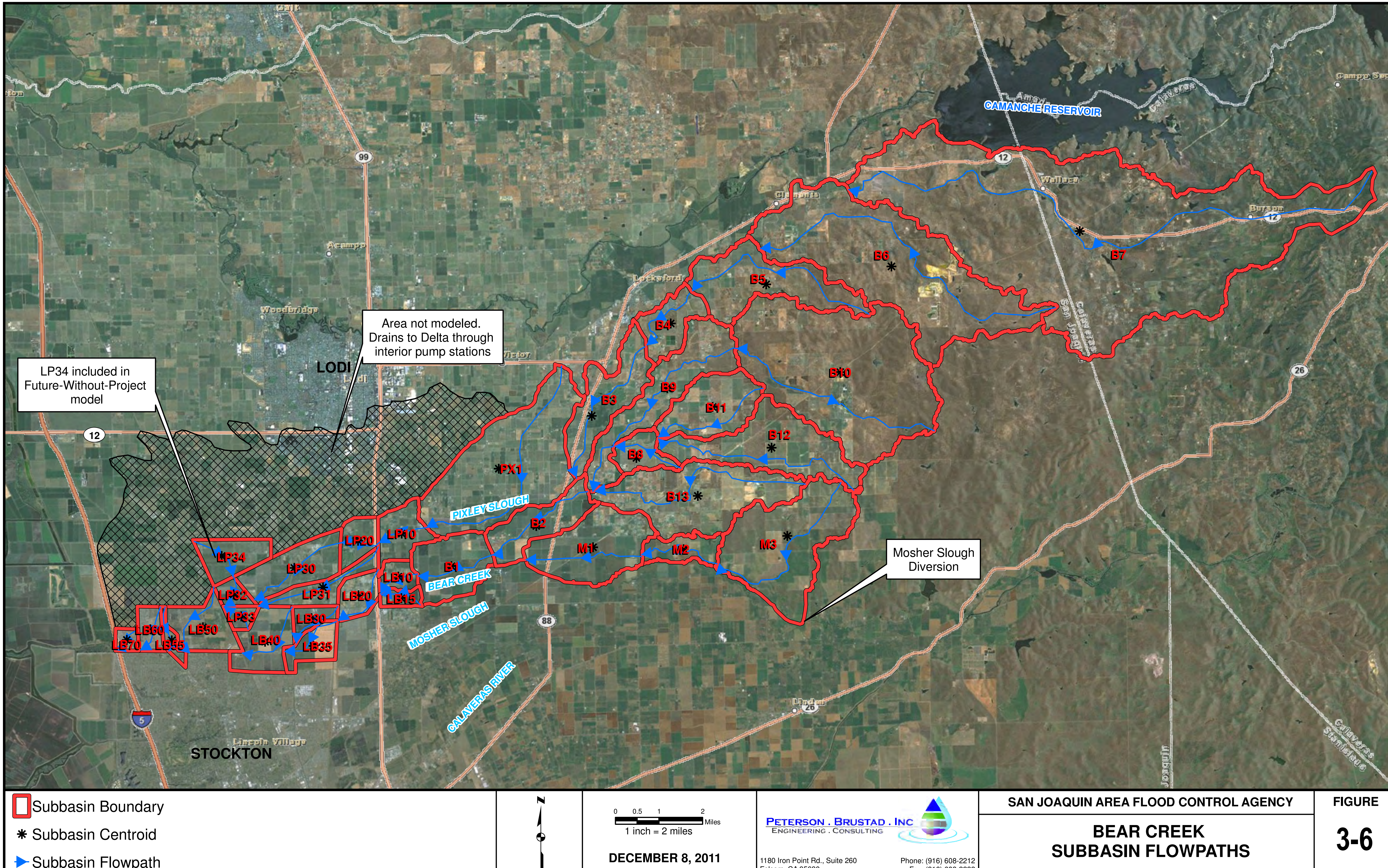
where:

Lg	=	Lag time [hours]
n	=	Average basin factor estimated using Figure E-2 in the San Joaquin County Hydrology Manual
L	=	Length of longest watercourse [miles]
L_C	=	Length of longest watercourse measured to the centroid of the basin [miles]
S	=	Overall slope of longest watercourse [feet/mile]

L, L_C, and S were calculated using ArcGIS software. Flowpaths identified for these calculations are shown in Figure 3- 6.

3.3.5. Channel Routing

The PBI Model utilizes the Muskingum-Cunge routing method to represent attenuation of flood waves within Bear Creek channels. Routing reach lengths and slopes were measured using ArcGIS software. Manning's n values and channel cross-sections were imported from the 1998 SJAFCA HEC-1 model[8].



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SAN JOAQUIN AREA FLOOD CONTROL AGENCY

FIGURE 3-6

BEAR CREEK SUBBASIN FLOWPATHS

The following table provides a summary of routing elements included in the PBI Model.

Table 3- 2. Summary of Bear Creek model routing elements.

Routing Element	Length	Slope	Manning's n Main Channel	Manning's n Overbank	Description From	Description To
	[ft]	[ft/ft]				
RB7	18,670	0.0012	0.045	0.06	B7	B6
RN1222	10,190	0.0018	0.045	0.06	B6	B5
RN1210	10,010	0.0008	0.035	0.05	B5	B4
RN1209	22,300	0.0011	0.035	0.05	B4/B3	B9
RB10	27,690	0.0014	0.040	0.55	B10	B9
RTHDR	10,880	0.0015	0.035	0.05	B11/B12	B8
RN1208	1,860	0.0022	0.035	0.05	B8	B9
RN1204	13,060	0.0010	0.030	0.04	B9	B2
RMSRTN	9,820	0.0014	0.030	0.04	B2	B1
RN1203	5,260	0.0009	0.030	0.04	B1	LB15/MSDIV
RM3	11,890	0.0014	0.045	0.06	M3	M2
RNM2	14,430	0.0014	0.045	0.06	M2	M1
RN1202	980	0.0010	0.030	0.04	B1/LB15	LB10
R1020	6,530	0.0011	0.030	0.04	LB10	LB20
R2030	6,380	0.0014	0.030	0.04	LB20	LB30
R3035	1,810	0.0050	0.030	0.04	LB30	LB35
R3540	4,690	0.0009	0.030	0.04	LB35	LB40
R4050	7,080	0.0018	0.030	0.04	LB40	LB50
RPX1	7,160	0.0007	0.050	0.06	PX1	LP10
RP1020	5,470	0.0007	0.050	0.06	LP10	LP20
RP2030	13,860	0.0010	0.050	0.06	LP20	LP30
RP313	4,200	0.0012	0.050	0.06	LP30	LP32/LP33
RP325	8,370	0.0013	0.050	0.06	LP32/LP33	LB50
R5055	1,960	0.0010	0.030	0.04	LB50	LB55
R5560	6,510	0.0011	0.030	0.04	LB55	LB60/LB70

Twenty-five reaches covering a total of approximately 44 miles of the Bear Creek stream system are included in the PBI Model.

3.3.6. Loss Rates

As discussed in Section 3.2.1, the 1998 SJAFCA HEC-1 model used the SCS Curve Number method to calculate loss rates. The PBI Model differs from the SJAFCA HEC-1 model in that it uses the initial and constant loss rate method to model subbasin losses.

The Natural Resources Conservation Service (NRCS) has classified all soils into four hydrologic soil groups (A, B, C, and D) according to their infiltration rates[15]:

Table 3- 3. NRCS hydrologic soil groups.

Hydrologic Soil Group	Loss Rate Range [in/hr]	PBI's Assumed Loss Rate[a] [in/hr]
A	> 0.30	0.35
B	0.15 - 0.30	0.2
C	0.05 - 0.15	0.1
D	0.00 - 0.05	0.025

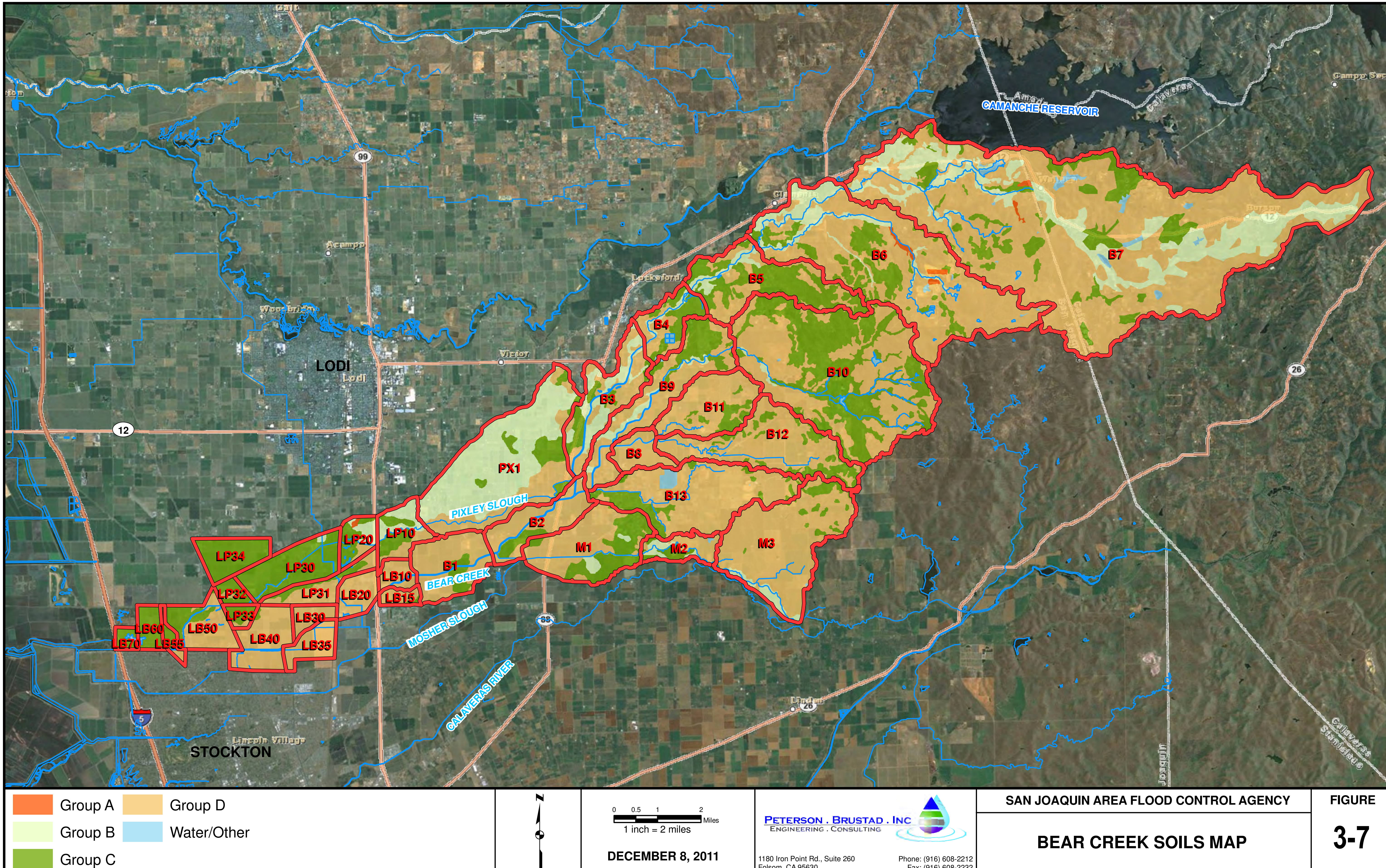
[a]This loss rate value was assigned to each soil group for initial calculations of composite loss rates. The calculated composite loss rates were then adjusted during the calibration process.

A GIS soils layer was obtained from the NRCS[13] and used to determine the proportional coverage of soil groups within Bear Creek subbasins (Figure 3- 7). NRCS GIS soils data was not available for Calaveras County. Soils data for this part of the study area was obtained from the Calaveras County Soil-Vegetation Survey[14]. A weighted average of loss rates was calculated for each subbasin and adjusted during the calibration process (See Section 3.4). After the calibration adjustment, subbasin loss rates range from 0.020 inches per hour to 0.118 inches per hour as shown in Attachment 3-C.

EM 1110-2-1417[18] recommends that initial losses are set between 0.5-1.5 inches for agricultural areas. Initial losses were set to 0.5 inches for all agricultural/rural subbasins in the foothills and to 1.5 inches for agricultural/rural subbasins in the valley. For urban subbasins, initial losses were set to 0.2 also based on guidelines listed in *EM 1110-2-1417*.

3.3.7. Impervious Percentages

Impervious percentages were assigned based on the extent of urbanization within each subbasin. Aerial photos including those contained within 2010 LiDAR datasets[4] were used to assess existing urbanization in the Bear Creek watershed. subbasins were classified into several categories with assigned impervious percentages as shown in Table 3- 4. The impervious percentages corresponding to each land use type were selected with the guidance of San Joaquin County's *Hydrology Manual*[10].


CAMANCHE RESERVOIR
LODI
STOCKTON
PIXLEY SLOUGH
BEAR CREEK
MOSHER SLOUGH
CALAVERAS RIVER
PX1
B1
B2
B3
B4
B5
B6
B7
B8
B9
B10
B11
B12
B13
M1
M2
M3
LP10
LP20
LP30
LP31
LP32
LP33
LP34
LB10
LB15
LB20
LB30
LB35
LB40
LB50
LB55
LB60
LB70
Group A
Group B
Group C
Group D
Water/Other
0 0.5 1 2 Miles
1 inch = 2 miles
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SAN JOAQUIN AREA FLOOD CONTROL AGENCY
BEAR CREEK SOILS MAP
FIGURE
3-7

Table 3- 4. Land use types and their corresponding impervious percentages.

Land Use Type	Impervious Percentage
Agricultural/Open Space	2%
Agricultural with Rural Residential Development	5%
Fully Developed Residential	60%

3.4. MODEL CALIBRATION

The 1998 SJAFCA HEC-1 model was calibrated using an annual exceedance probability plot at the Lockeford stream gage[8]. This plot was used to determine a 1/100 AEP flow event and it was assumed that a 1/100 AEP rainfall event would produce a 1/100 AEP streamflow event.

The PBI Model was calibrated to an observed rainfall-runoff event using gaged data retrieved from San Joaquin County's ALERT System[16]. Three gages were used for the model calibration. The Bear Creek streamflow gage (ALERT Gage 238) and Alpine Road rainfall gage (ALERT Gage 239) are both located on Bear Creek between Highway 99 and State Route 88 (see Figure 3- 2). In addition, the Robidart Ranch gage (ALERT Gage 237) provides rainfall data for the subbasins in the upper portion of Bear Creek watershed.

The storm selected to calibrate the PBI Model was the largest event recorded by the Bear Creek gage and is approximately a 1/10 AEP event. The rainfall event took place between January 29, 1998 and February 9, 1998 (12-day duration) and totaled 6.26 inches. The "effective" portion of the storm included 2.88 inches of rainfall falling in 32 hours and was responsible for the peak streamflow seen on February 3, 1998.

The Bear Creek gage location corresponds to Model Element MSRTN. During the calibration process, constant loss rates were adjusted to match the PBI Model's hydrograph at MSRTN to observed streamflow records from the Bear Creek gage. Constant loss rates were initially calculated based on the makeup of soils in each subbasin (see Section 3.3.6). The loss rates were then adjusted by a factor of 0.80 during the calibration process. The results of the calibration are shown in Figure 3- 8.

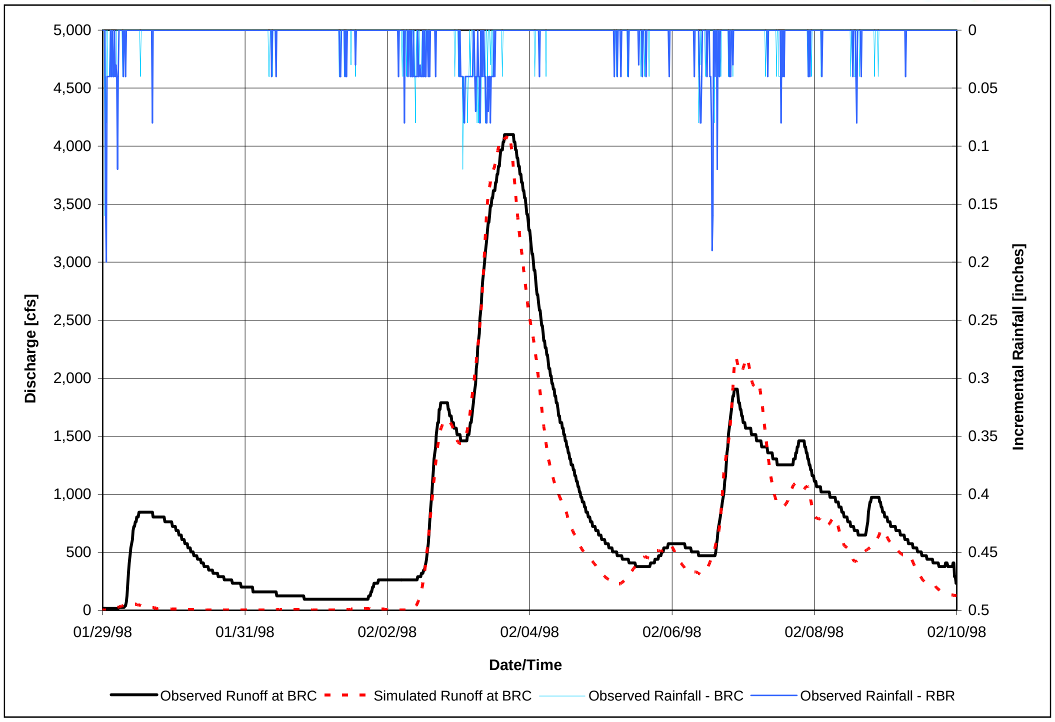


Figure 3- 8. Observed versus modeled flow for the Bear Creek calibration event.

At the onset of the storm, the initial runoff response is not picked up by the HEC-HMS model. This is due to the initial loss parameter being set to1.5 inches for the pervious areas of all subbasins (see Section 3.3.6). The subbasins upstream of the BRC gage are undeveloped and contain almost entirely pervious surfaces which are affected by the initial loss parameter. Although this runoff response could be captured by decreasing the initial losses, initial loss was held at 1.5 inches based on the ranges suggested in the Comp Study[9] and the variability in tilling practices, which have a major impact on initial losses. The emphasis of the hydrologic analysis is on peak event estimation, however, which is relatively insensitive to initial loss assumptions.

3.5. DEVELOPMENT CONDITIONS

3.5.1. Existing Conditions

An ‘Existing Conditions’ model run was performed to evaluate peak flows given current (2010) land use and hydrologic conditions within the Bear Creek watershed. Subbasin S-graphs, ‘n’ values, and impervious percentages were set according to current land cover conditions using field knowledge supplemented by aerial photos.

In general, the upstream watershed consists of natural or agricultural land whereas the lower portions of Bear Creek watershed are developed areas in and around the city of Stockton. A summary table of the subbasin characteristics used for ‘Existing Conditions’ model runs is provided in Attachment 3-D.

As seen in Figure 3- 9, subbasins LB50, LB55, LB60, LB70, and LP33 are considered to be developed and flows from these basins are directed through three storm water pump stations: the Spanos Park-I-5 pump station (PLB6070), the Thornton pump station (PLB5055), and the Pixley pump station (PLP33). The pump stations discharge flows up to their design capacities (see Section 3.3.2) into Bear Creek and Pixley Slough. Any subbasin flows exceeding pump station capacities would result in temporary ponding within the subbasin. This ponding would be entirely due to inadequate pump capacities and would be independent of exterior stage conditions in the receiving stream.

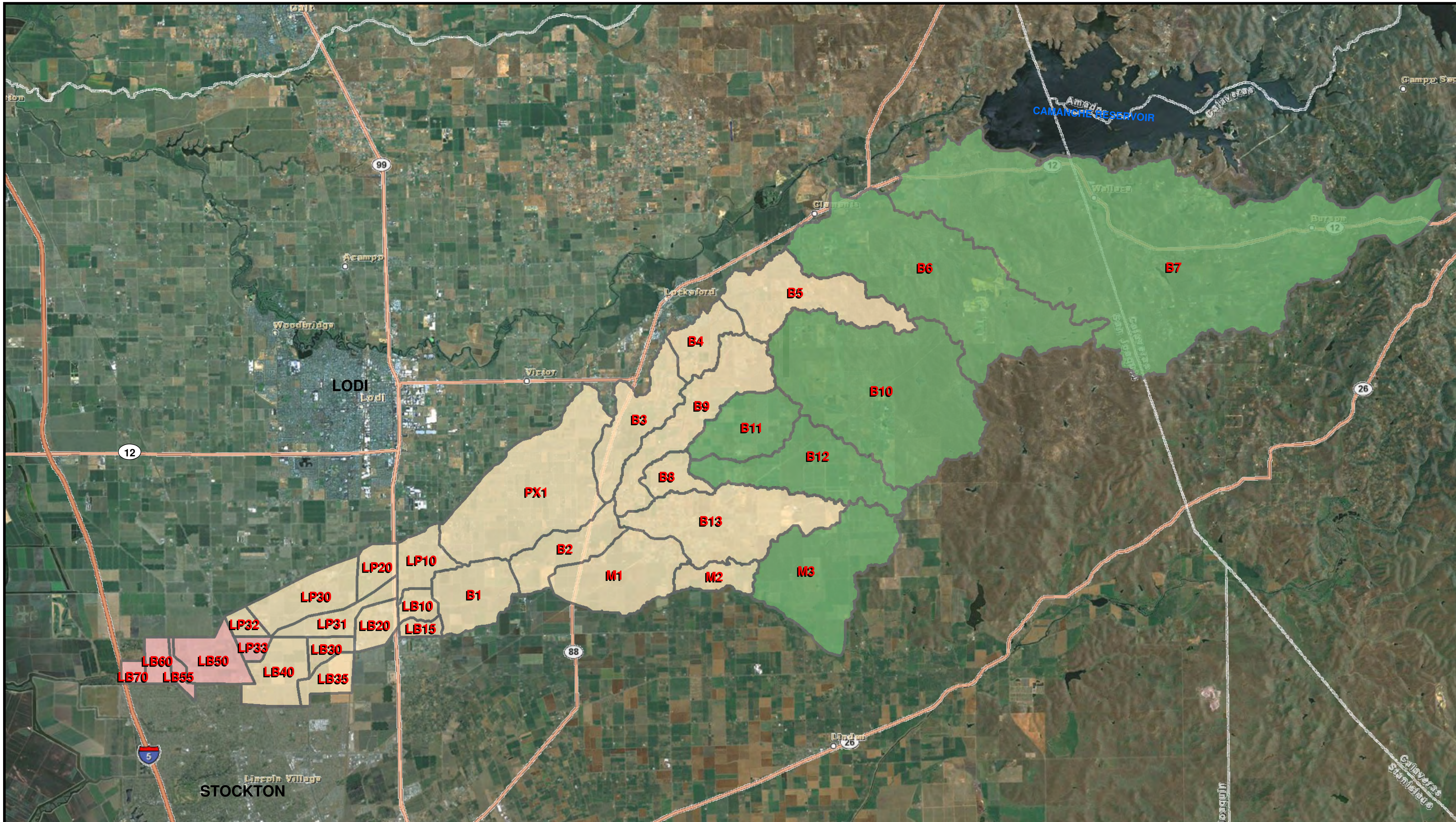
3.5.2. Future-Without-Project Conditions

A 'Future Conditions' model run was performed to evaluate peak flows for estimated future (2070) land use and hydrologic conditions within the Bear Creek watershed. Land use conditions are based on the City of Stockton 2035 General Plan[12] and the San Joaquin County General Plan[17].

As shown in Figure 3- 10, the upstream watershed remains unchanged and consists of natural or agricultural land whereas the lower portions of Bear Creek watershed experience an increase in development. The following 9 subbasins were previously undeveloped in the 'Existing Conditions' model and would be developed for the 'Future-Without-Project Conditions' model: LB10, LB15, LB20, LB30, LB35, LB40, LP30, LP31, LP32. As previously mentioned, subbasin LP34 was added to the 'Future-Without-Project' model to account for added drainage area that is expected to be pumped into Bear Creek.

In addition to updating subbasin S-graphs, 'n' values, and impervious percentages for the newly developed areas, storm water pump stations were also added to these subbasins. As previously mentioned, flows exceeding pump station capacities would cause temporary ponding, which was assumed to be mitigated within the subbasin through on-site detention.

A summary table of subbasin characteristics used for 'Future-Without-Project Conditions' model runs is provided in Attachment 3-E.



Foothill
Valley Undeveloped
Valley Developed

N

0 0.5 1 2 Miles
1 inch = 2 miles

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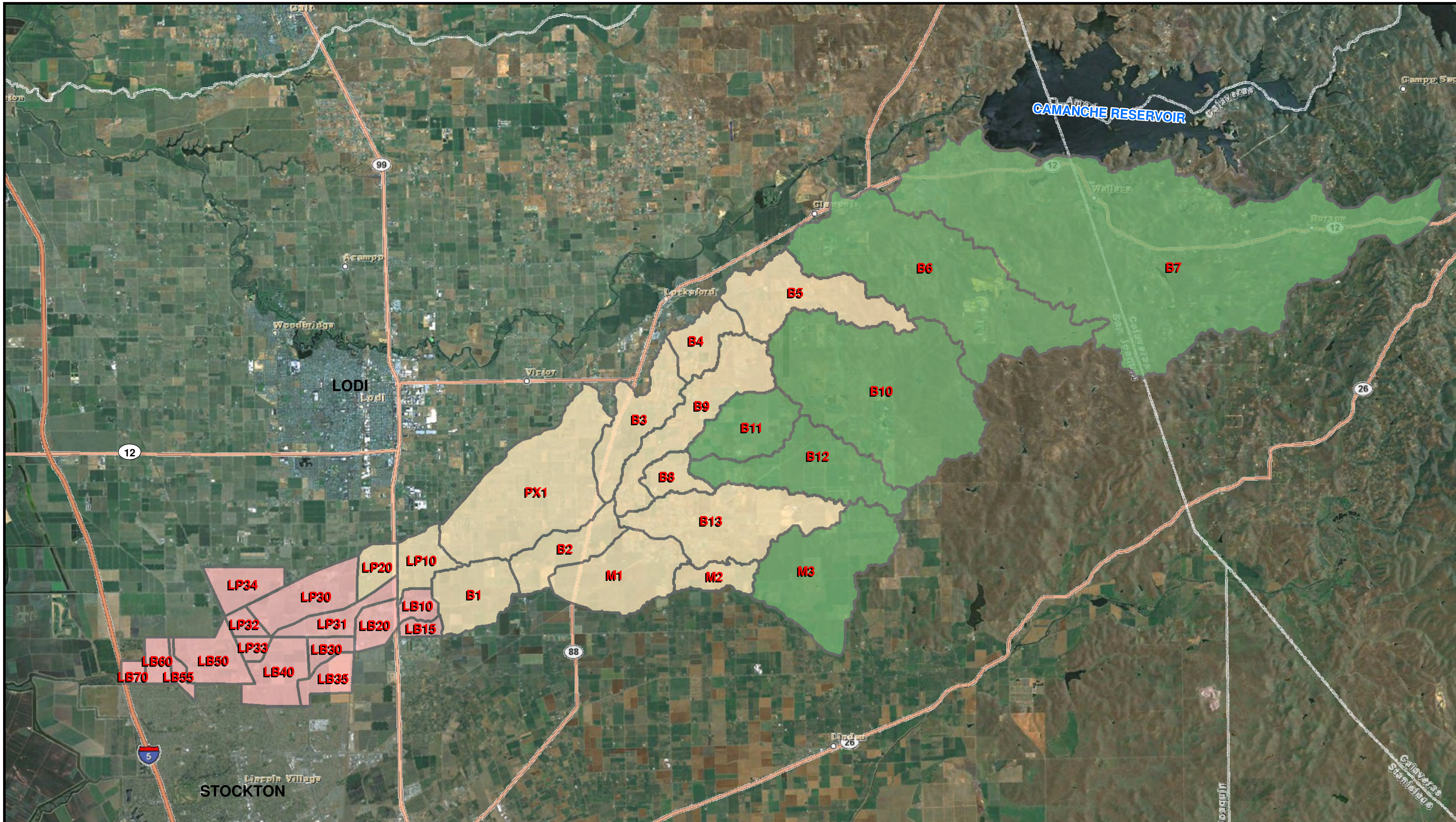
EXISTING DEVELOPMENT CONDITIONS FOR BEAR CREEK WATERSHED

FIGURE 3-9



- Valley Developed
- Valley Undeveloped
- Foothill

0 0.5 1 2 Miles

1 inch = 2 miles

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SAN JOAQUIN AREA FLOOD CONTROL AGENCY

FIGURE

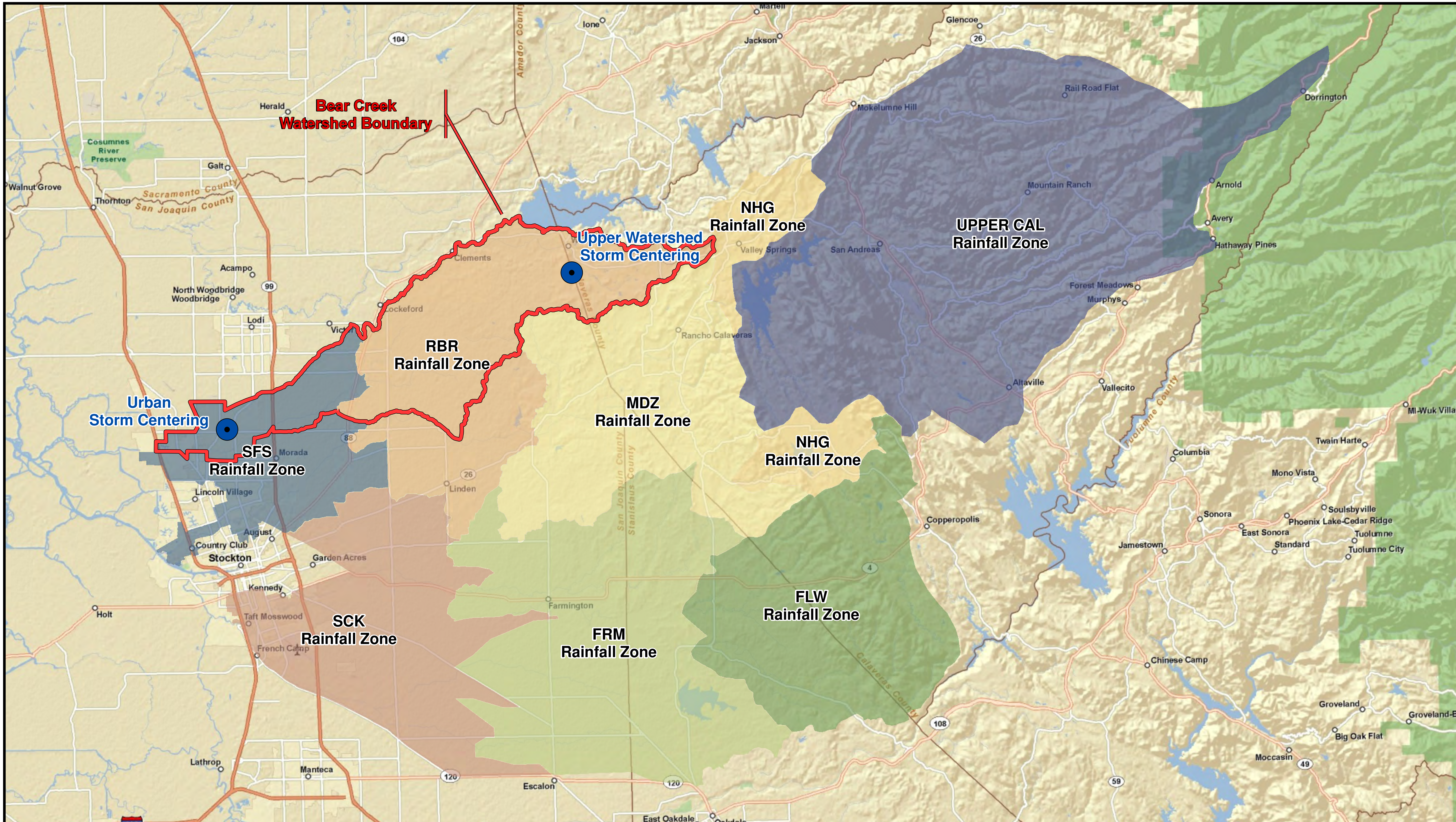
3-10

FUTURE DEVELOPMENT CONDITIONS FOR BEAR CREEK WATERSHED

3.6. STORM CENTERINGS

Two storm centerings were analyzed for the Bear Creek watershed (Figure 3- 11). One centering was placed over the upper portion of the watershed to create high flows in the tributary channels and concurrent inputs to the lower channel. The second centering was for interior drainage purposes and was placed over the urban areas of the watershed. The 8 AEP storm frequencies were analyzed for each centering. This selection of design storms provides a wide range of scenarios that can be used for planning purposes.

Calculated area reduction factors and resulting area-reduced rainfall depths for each rainfall zone are provided in Attachment 3-F for all frequency-duration-storm centering combinations.



N

0 5 Miles

1 : 300,000

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LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

BEAR CREEK WATERSHED STORM CENTERINGS

FIGURE 3-11

3.7. MODEL SIMULATIONS

Bear Creek production runs include 32 scenarios with unique combinations of development conditions, storm frequencies, and storm centerings.

Table 3- 5. Bear Creek production run scenarios.

Development Conditions	Storm Centerings	AEP Events
Existing Conditions	Urban	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	Upper Watershed	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
Future-Without-Project Conditions (2070)	Urban	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	Upper Watershed	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500

3.7.1. Summary of Results

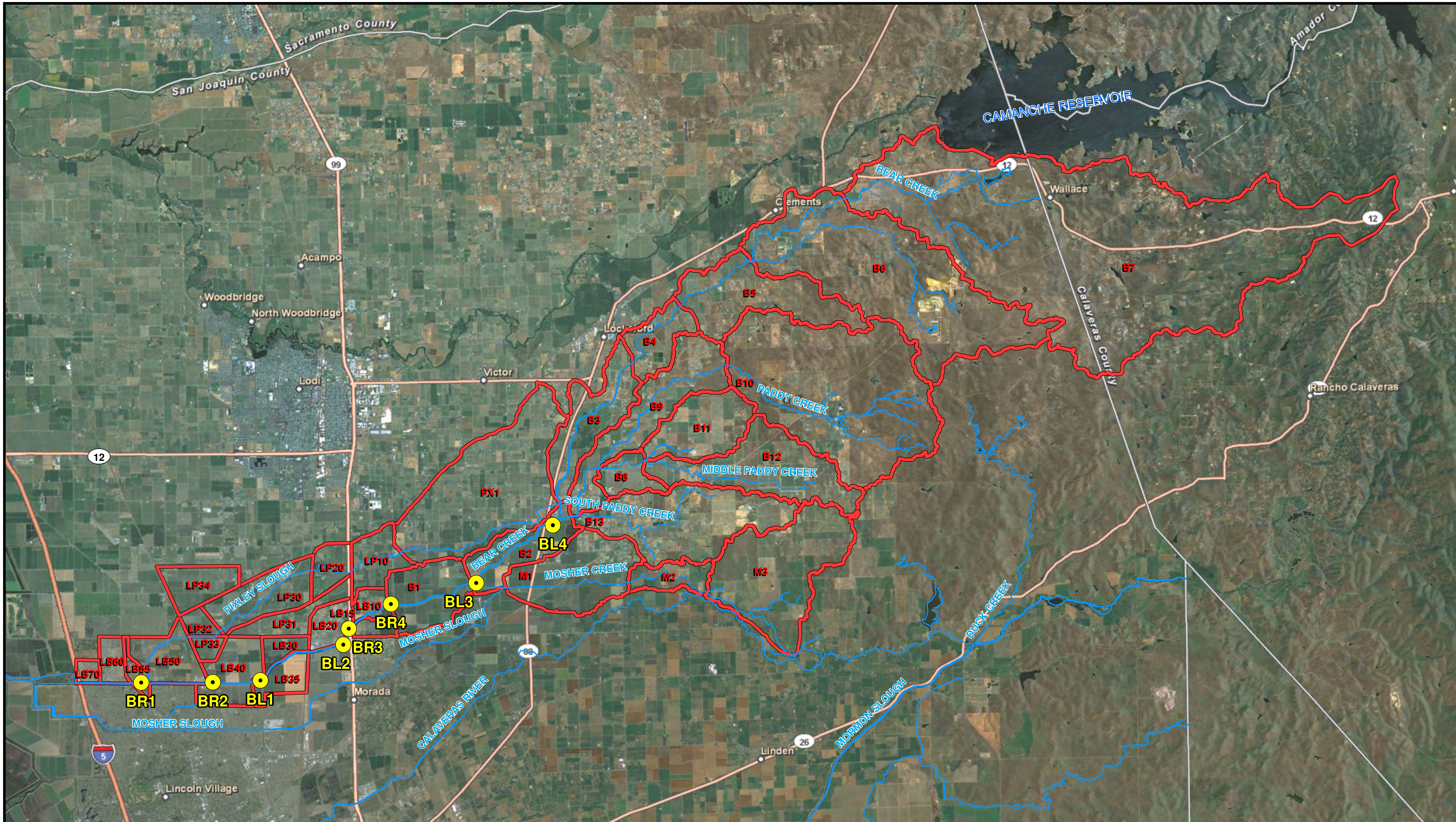
Peak flow results were extracted from HEC-HMS at each LSJRFS index point. Locations of LSJRFS index points within the Bear Creek watershed are shown in Figure 3- 12. Table 3- 6 and Table 3- 7 summarize peak flows for 'Existing Conditions' runs and for 'Future-Without-Project Conditions' runs, respectively.

In all cases, peak flows from the Upper Watershed storm centering scenario are higher than the urban storm centering scenario. The Upper Watershed centering is therefore the controlling scenario for the LSJRFS.

3.7.2. Uncertainty Parameters

For the purposes of the LSJRFS, uncertainty parameters for each flow-frequency dataset can be estimated within HEC-FDA during the project's economic analysis. HEC-FDA defines uncertainty in terms of confidence intervals or standard deviations given inputs of flow-frequency data (provided in Table 3- 6 and Table 3- 7) and an equivalent record length.

The equivalent record length is an estimate of the overall "worth" or "quality" of the flow-frequency function, expressed as the number of years-of-record[19]. For probability functions derived at ungaged locations using model or other data, the equivalent record length is based on a judgment of the quality of that model or data. EM 1110-2-1619[20] provides guidelines for assigning equivalent record lengths and estimates that a rainfall-runoff model calibrated to an observed event at a short-interval runoff gage has an <u>equivalent record length of 20-30 years</u>.



LSJRFS Index Point

Subshed Boundary

N

0 0.5 1 2 Miles

1 inch = 2 miles

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LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

BEAR CREEK WATERSHED INDEX POINTS

FIGURE

3-12

Table 3-6. Peak Flow Results for Bear Creek - Existing Conditions [cfs]

LSJRFS Index Point ID	Description	Urban Storm Centering								Upper Watershed Storm Centering							
		1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP	1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP
BL4	Bear Creek near Hwy 88	1,520	2,290	2,850	3,630	4,250	4,900	5,520	6,510	1,900	2,680	3,300	4,180	4,890	5,560	6,320	7,410
BL3	Bear Creek at Alpine Rd.	1,660	2,510	3,150	4,110	4,940	5,790	6,650	7,850	2,060	2,940	3,630	4,810	5,710	6,620	7,570	8,880
BR4	Bear Creek near CCTRR	1,670	2,540	3,190	4,190	5,030	5,890	6,760	7,990	2,060	2,940	3,670	4,850	5,770	6,680	7,650	8,970
BR3	Bear Creek at Hwy 99	1,670	2,540	3,190	4,230	5,070	5,930	6,810	8,040	2,060	2,940	3,690	4,870	5,790	6,700	7,670	9,000
BL2	Bear Creek d/s of Eight Mile Rd.	1,670	2,550	3,200	4,270	5,110	5,980	6,850	8,090	2,050	2,940	3,700	4,900	5,810	6,730	7,700	9,030
BL1	Bear Creek near West Ln.	1,680	2,570	3,250	4,340	5,200	6,070	6,960	8,200	2,050	2,950	3,740	4,950	5,870	6,800	7,780	9,110
BR2	Bear Creek at UPRR	1,690	2,580	3,310	4,430	5,300	6,190	7,080	8,340	2,050	2,960	3,790	5,020	5,940	6,880	7,870	9,210
BR1	Bear Creek d/s of Pixley Slough confl.	1,720	2,670	3,520	4,810	5,810	6,800	7,800	9,190	2,080	2,990	3,840	5,180	6,200	7,240	8,340	9,820
D2	Bear Creek at I-5	1,760	2,710	3,600	4,900	5,920	6,960	7,990	9,430	2,110	3,020	3,890	5,270	6,340	7,400	8,490	10,000

Table 3-7. Peak Flow Results for Bear Creek - Future Conditions [cfs]

LSJRFS Index Point ID	Description	Urban Storm Centering								Upper Watershed Storm Centering							
		1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP	1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP
BL4	Bear Creek near Hwy 88	1,520	2,290	2,850	3,630	4,250	4,900	5,520	6,510	1,900	2,680	3,300	4,180	4,890	5,560	6,320	7,410
BL3	Bear Creek at Alpine Rd.	1,660	2,510	3,150	4,110	4,940	5,790	6,650	7,850	2,060	2,940	3,630	4,810	5,710	6,620	7,570	8,880
BR4	Bear Creek near CCTRR	1,670	2,540	3,190	4,190	5,030	5,890	6,760	7,990	2,060	2,940	3,670	4,850	5,770	6,680	7,650	8,970
BR3	Bear Creek at Hwy 99	1,680	2,550	3,200	4,250	5,090	5,950	6,810	8,070	2,070	2,960	3,710	4,890	5,820	6,730	7,700	9,010
BL2	Bear Creek d/s of Eight Mile Rd.	1,690	2,590	3,260	4,340	5,180	5,980	6,910	8,160	2,070	2,970	3,740	4,920	5,860	6,790	7,790	9,100
BL1	Bear Creek near West Ln.	1,700	2,590	3,300	4,430	5,250	6,080	7,020	8,280	2,080	2,980	3,790	5,000	5,920	6,900	7,870	9,230
BR2	Bear Creek at UPRR	1,740	2,630	3,320	4,540	5,390	6,210	7,230	8,470	2,110	3,020	3,840	5,050	6,070	7,030	7,960	9,380
BR1	Bear Creek d/s of Pixley Slough confl.	1,910	2,790	3,780	5,060	6,320	7,260	8,210	9,460	2,170	3,070	4,050	5,470	6,600	7,750	8,810	10,410
D2	Bear Creek at I-5	2,000	2,830	3,840	5,210	6,440	7,440	8,350	9,710	2,200	3,100	4,140	5,600	6,730	7,910	8,990	10,560

4.0 MOSHER SLOUGH HEC-HMS MODELING

4.1. GENERAL

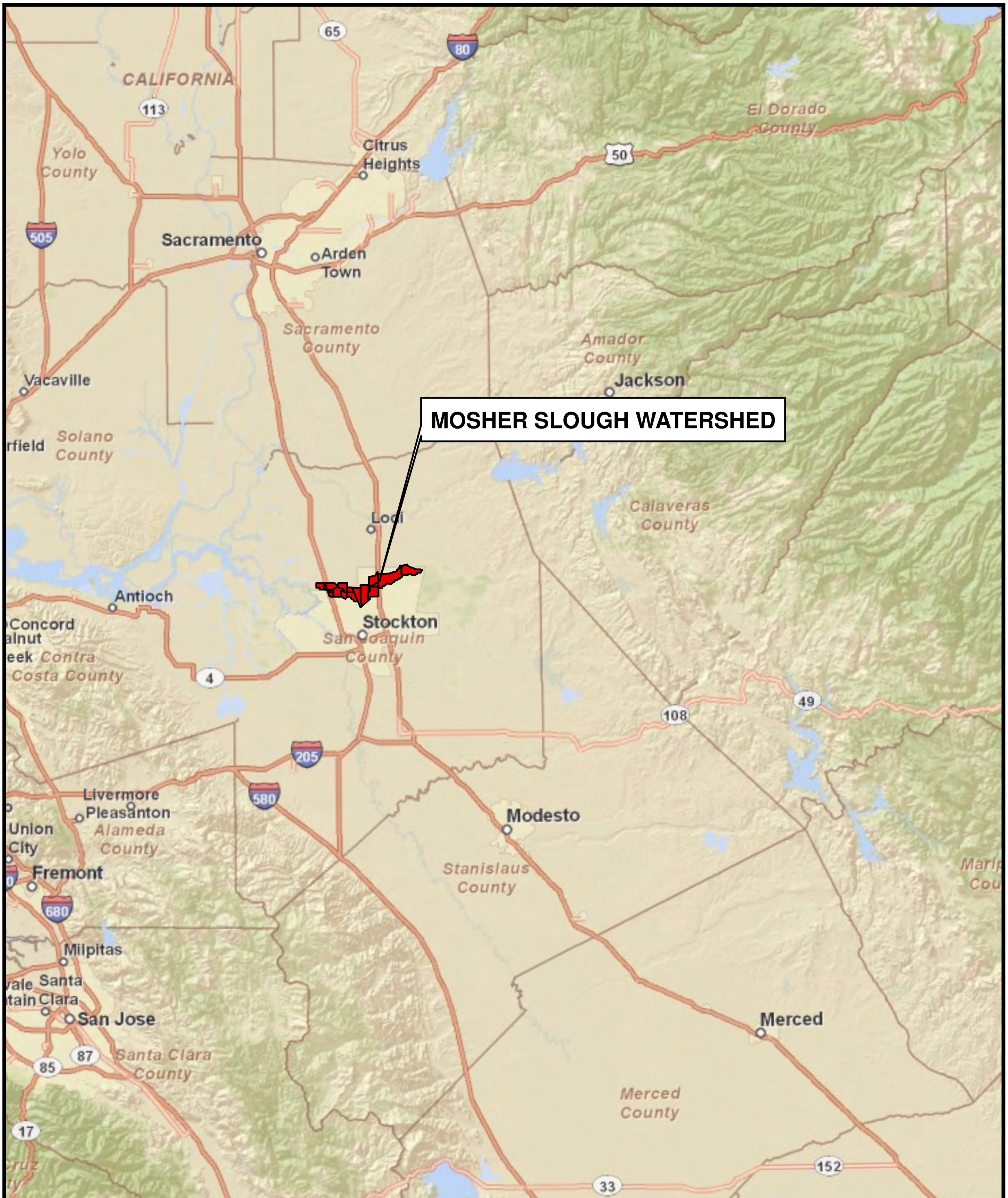
4.1.1. Location

Mosher Slough is located near the city of Stockton in San Joaquin County, California (Figure 4- 1). The majority of the watershed is located in the urbanized area of Stockton between Interstate-5 and Highway 99 with the watershed area totaling approximately 16 square miles. The watershed's terrain has moderate slopes and reaches a maximum elevation of 65 feet above the modeled outlet at the confluence of Mosher Slough and Bear Creek just west of Interstate-5.

The HEC-HMS model described in this report includes only the lower portion of Mosher Slough which begins immediately below the diversion that routes the entirety of Upper Mosher Creek to Bear Creek (see Figure 4- 2). The hydrology for Upper Mosher Creek is included in the Bear Creek HEC-HMS model as described in Section 3.0 of the LSJRFS Hydrology Report.

4.1.2. Topography

The HEC-HMS model utilized for this study is titled the PBI Mosher Slough Model (PBI Model) which is georeferenced to the NAD 1983 State Plane California Coordinate System Zone III (U.S. Survey Feet). Vertical elevations are reported in the NAVD 1988 datum. Topography used for model development included United States Geological Survey (USGS) 30-meter Digital Elevation Models (DEMs)[3]. Department of Water Resources (DWR) LiDAR data[4] was also used to confirm subbasin boundaries (State vertical datum in NAVD 88).



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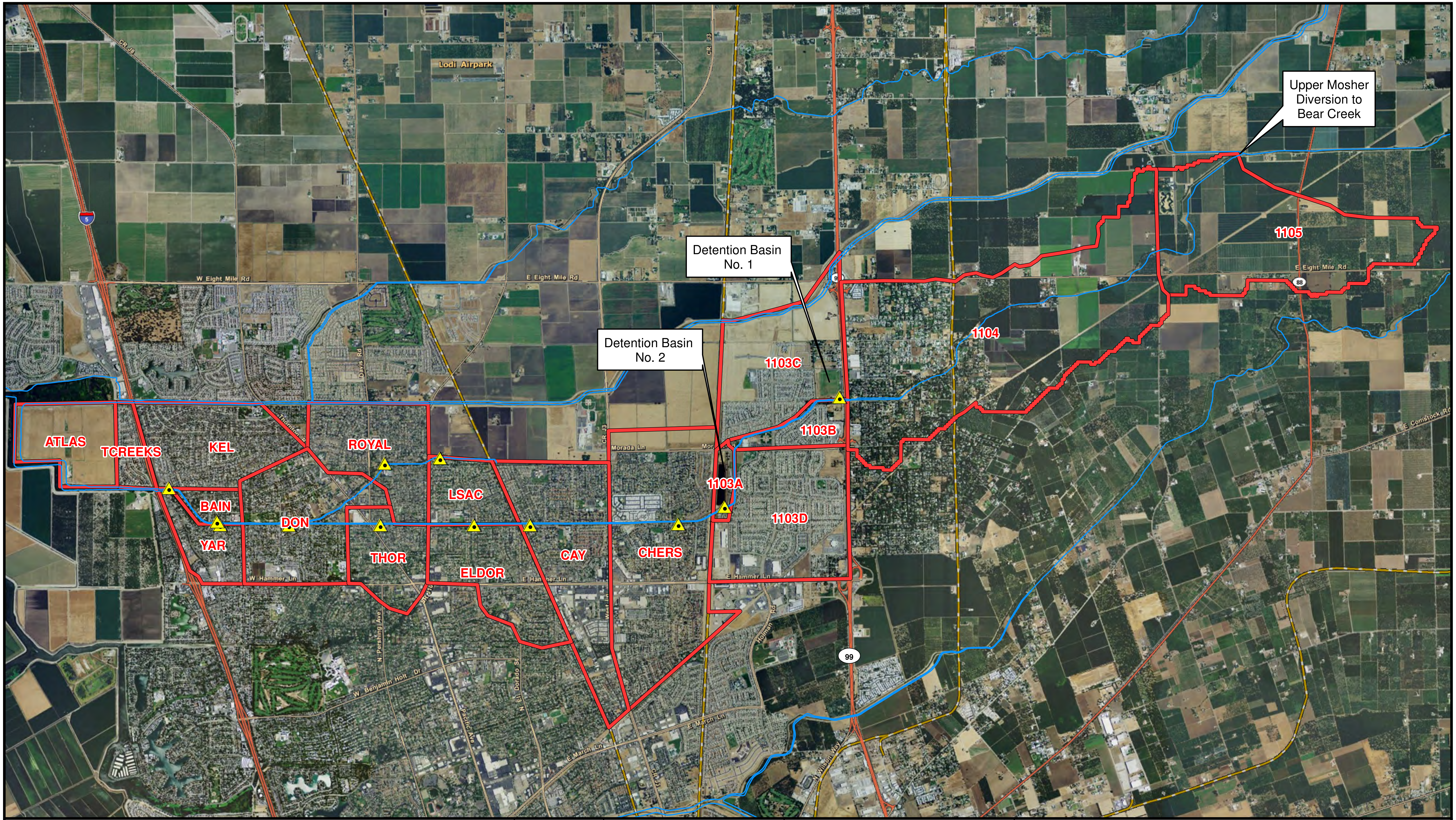
SAN JOAQUIN AREA FLOOD CONTROL AGENCY

FIGURE 4-1

VICINITY MAP OF THE MOSHER SLOUGH STUDY AREA



Subbasin Boundary

Existing Pump Station

N

0 1,000 2,000 4,000 Feet

1 inch = 4,000 feet

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MOSHER SLOUGH
HEC-HMS SUBBASINS

FIGURE

4-2

4.2. MODEL DEVELOPMENT

The PBI model was developed by converting the 1998 SJAFCA HEC-1 into HEC-HMS format using HEC-HMS version 3.4[5] and HEC-GeoHMS version 4.2[6]. A summary of the tasks performed are listed below:

1. The 1998 SJAFCA HEC-1 model was imported into HEC-HMS (See Section 4.2.2).

2. Subbasin boundaries from SJAFCA HEC-1 model were updated using HEC-GeoHMS and United States Geological Survey (USGS) Digital Elevation Models (DEMs) (See Section 4.3.1).

3. Pump stations were coded into the PBI model based on design pumping rates provided by the City of Stockton[7] (See Section 4.3.2).

4. New diversions and channel routing parameters were coded into the PBI Model (See Sections 4.3.3 and 4.3.5, respectively), replacing those used in the SJAFCA HEC-1 model.

5. New loss rates and impervious percentages were coded into the PBI Model (See Section 4.3.6 and Section 4.3.7) replacing those used in the SJAFCA HEC-1 model.

6. S-graphs and lag times were assigned to each subbasin (See Section 4.3.4).

7. The PBI Model was set up to simulate both Existing (Section 4.5.1) and Future-Without-Project (Section 4.5.2) scenario runs.

4.2.1. SJAFCA HEC-1 Model

The PBI Model is a conversion and update of the HEC-1 model developed for SJAFCA by HDR Engineering, Inc. in 1998[8].

The 1998 SJAFCA HEC-1 model utilized the San Joaquin County LA preprocessor to convert S-graphs to unit hydrographs for each subbasin. Two types of S-graphs were obtained from the San Joaquin County Hydrology Manual[10] and used based on the surface condition classification of the subbasin: Valley Undeveloped and Valley Developed. Lag times were calculated by HDR using basin 'n' values, length of subbasin flow, flow length from the centroid, and slope of the basin.

The SJAFCA HEC-1 model used the SCS curve number method to account for subbasin losses. Curve numbers typically ranged from 81 to 86 depending on soil type and cover. Attachment 4-A lists the parameters used in the 1998 SJAFCA HEC-1 model.

4.2.2. Conversion from HEC-1 to HEC-HMS

The 1998 SJAFCA HEC-1 model was successfully imported into HEC-HMS as the fundamental basis for the PBI Model.

Certain features in the HEC-1 software are not supported in HEC-HMS and therefore were not properly transferred during the import process. Pump station data and meteorological data from the SJAFCA HEC-1 model were manually coded into the PBI Model so as to conform to HEC-HMS formatting.

In addition, there are computational differences between the HEC-1 and the HEC-HMS software. One such difference involves the Muskingum-Cunge stream segment routing technique used for the PBI model. In HEC-HMS, channel properties are computed based on the physical characteristics of that channel, whereas in HEC-1 the properties are computed with formulas based on a kinematic wave assumption[5]. This causes minor differences in the flows that are transferred through the routing parameters. HEC-HMS results are preferred because of the refined computational techniques that have been implemented.

For initial PBI Model testing, user-specified hyetographs were assigned to each subbasin based on 1/100 AEP storm data defined in the 1998 SJAFCA HEC-1 model's input files. This storm event was run for debugging purposes and results were made sure to match the SJAFCA HEC-1 model results.

4.3. MODEL FEATURES

The 1998 SJAFCA HEC-1 model was converted and modified for this study to form the PBI Mosher Slough HEC-HMS Model. The PBI Model components are described in the following sections.

4.3.1. Subbasins

Subbasin boundaries used in the 1998 SJAFCA HEC-1 model were cross-checked with USGS 30-meter DEM datasets[3] and modified where appropriate. Any boundary modifications were made using the ArcHydro and HEC-GeoHMS[6] extensions within the ArcGIS software package. These tools utilize geospatial data to interpret drainage patterns and delineate watershed boundaries accordingly. Where available, DWR LiDAR[4] data was used to confirm subbasin boundaries. For the majority of the watershed, west of Highway 99, subbasin boundaries were based on the City of Stockton's *Conceptual Storm Drain Master Plan*[11]. This portion of the watershed is urbanized and the boundaries from the City of Stockton take into account drainage improvements that have been made in the area.

The PBI model contains two additional subbasins when compared to the 1998 SJAFCA HEC-1 Model. The Twin Creeks and Atlas tracts, totaling 0.68 square miles, drain to Mosher Slough and are located just west of Interstate-5. Mosher Slough subbasins included in the PBI Model are shown in Figure 4- 2.

The PBI Model contains a total of 18 subbasins with drainage areas ranging from 0.17 square miles to 3.25 square miles with a total watershed area of approximately 16 square miles.

The GIS horizontal coordinates for each subbasin were used to georeference model elements within the PBI HEC-HMS Model. The subbasin GIS shapefile was inserted into the PBI Model as a background map.

4.3.2. Detention Basins and Pump Stations

The Mosher Slough system includes two detention basins that are intended to help reduce peak flows. 'Detention Basin No. 1' is located just west of Highway 99 on the north side of the main channel. It is connected to the main channel through a lateral weir that induces split flow for channel flows in excess of 230 cfs, with overflows into Detention Basin No. 1. The detained flows are held until the storm peak passes and then pumped back into Mosher Slough. Any inflow that causes Detention Basin No. 1 to exceed its 160 AF capacity is redirected back into the main channel. This is accomplished in HEC-HMS by connecting the main channel to a diversion element which directs any flow in excess of 230 cfs to a reservoir element. This reservoir element represents the detention pond and is coded with a spillway to take any flow exceeding the 160 AF pond capacity and spill it back into the main channel. In subsequent LSJRFS tasks, HEC-RAS runs will better model flow split hydraulics.

Detention Basin No. 2 and pump station are located just upstream from the formerly named Southern Pacific Railroad. This detention basin collects all runoff from subbasins 1103A, 1103B, 1103C, and 1103D. The pump station pumps runoff stored in Detention Basin No. 2 and includes one pump at 10 cfs and an additional three pumps at 25.1 cfs each. During a flow event at or exceeding the 1/100 AEP, however, only the 10 cfs pump is activated while the other three pumps are not utilized until the event has subsided. Any flow that causes the detention basin to exceed its 265 AF capacity will cause a temporary backup of the storm sewer system until the 25.1 cfs pumps activate and drain the pond after the storm peak passes.

Along with the pumps at Detention Basin No. 2, eleven additional pump stations were included in the 'Existing Conditions' PBI Model to represent storm drainage conveyance from developed subbasins to Mosher Slough. Multiple pumps are included at each pump station with capacities assigned based on City of Stockton records. All pumps are set to discharge over the top of the levees and into the receiving channel above the highest stage expected. The exterior and interior areas at the pump stations are independent from one another.

One pump station was added into the 'Future-Without-Project Conditions' model for the Atlas Tract subbasin which is just downstream of I-5. This area is expected to become developed according to the City of Stockton 2035 General Plan[12]. Pump capacity was assigned at a rate of 0.37 cfs per acre of tributary area. This rate is based on the average flow rates of existing pump stations within the City of Stockton's systems.

Table 4- 1 provides a summary of pump stations included in the PBI Model.

Table 4- 1. Summary of Mosher Slough pump stations.

Pump Station Name	Contributing Subbasin Area [Sq. Mi.]	Pump Station Status	Pump Station Capacity [cfs]	Pump Station Notes
Cherbourg	1.78	Existing	199.5	1 @ 9 cfs 3 @ 63.5 cfs
Cayuga	1.17	Existing	269.2	4 @ 67.3 cfs
El Dorado	0.71	Existing	188.5	4 @ 46 cfs 1 @ 4.5 cfs
Thornton	0.47	Existing	26.8	2 @ 13.4 cfs
Lower Sacramento Rd.	0.35	Existing	19.0	1 @ 13.4 cfs 1 @ 5.6 cfs
Royal Oaks	0.73	Existing	204.5	1 @ 6.7 cfs 1 @ 44.6 cfs 2 @ 76.6 cfs
Don Avenue	0.96	Existing	77.7	1 @ 66.8 cfs 1 @ 10.9 cfs
Yarmouth	0.30	Existing	82.1	1 @ 7.8 cfs 1 @ 74.3 cfs
Bainbridge	0.14	Existing	43.5	3 @ 13.4 cfs 1 @ 3.3 cfs
Kelly	0.79	Existing	152.6	1 @ 8.9 cfs 1 @ 47.9 cfs 1 @ 45.7 cfs 1 @ 50.1 cfs
La Morada (Detention Basin No. 2)	8.30	Existing	85.3	1 @ 10 cfs 3 @ 25.1 cfs
Twin Brooks at Twin Creeks	0.17	Existing	34.8	3 @ 11.6 cfs
Atlas	0.51	Future	120.8	Based on 0.37 cfs per acre

4.3.3. Diversions

There is one diversion included in the PBI Mosher Slough model used to represent the lateral weir that diverts excess flows to 'Detention Pond No.1' located just west of Highway 99. This weir allows flows exceeding 230 cfs to overflow into the basin thereby regulating flows coming from upstream subbasins 1104 and 1105.

All flows from upper Mosher Creek are diverted to the main stem of Bear Creek at a location just upstream of the Central California Traction Railroad. Because this structure diverts all flow, Upper Mosher Creek was coded as a tributary area to Bear Creek and included in the PBI Bear Creek HEC-HMS model. The diversion was originally constructed by the United

States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS), and improved by SJAFCA in 1998.

4.3.4. S-graphs and Lag Times

As discussed in Section 4.2.1, the 1998 SJAFCA HEC-1 model utilized the San Joaquin County LA preprocessor for converting S-graphs to unit hydrographs. The PBI Model assigns Valley Undeveloped and Valley Developed S-graphs directly into HEC-HMS for each subbasin based on its location. S-graph data points were obtained from the San Joaquin County Hydrology Manual[10]. The S-graphs were developed based on rainfall-runoff data from Southern California catchments considered to be hydrologically similar to the local catchments.

Figure 4- 3and Figure 4- 4 show the time versus discharge relationship for each S-graph.

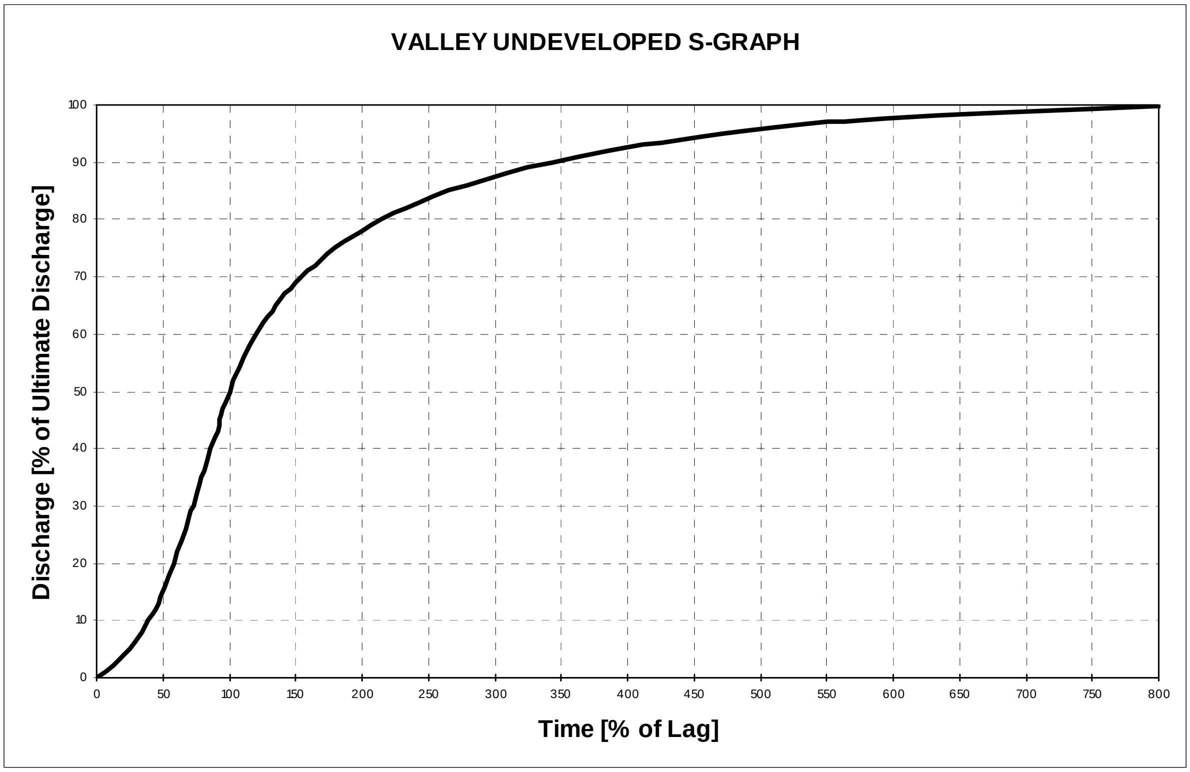


Figure 4- 3. San Joaquin County Valley Undeveloped S-graph

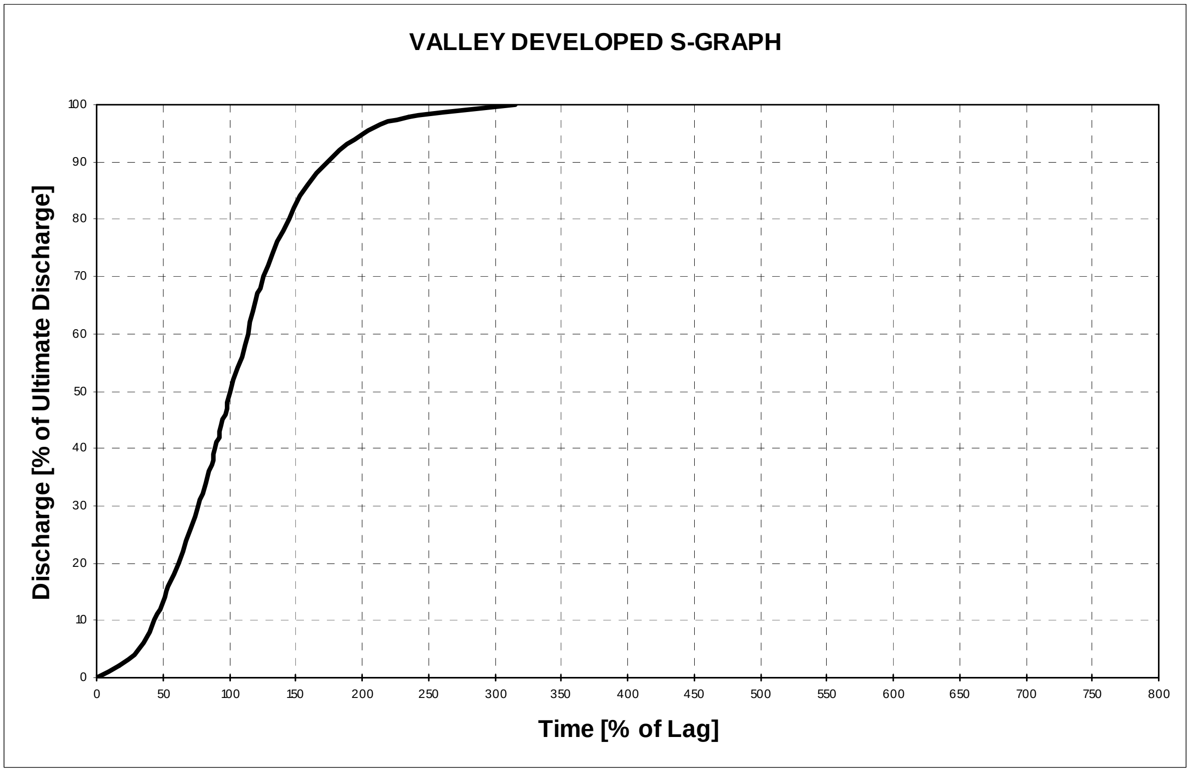


Figure 4- 4. San Joaquin County Valley Developed S-graph

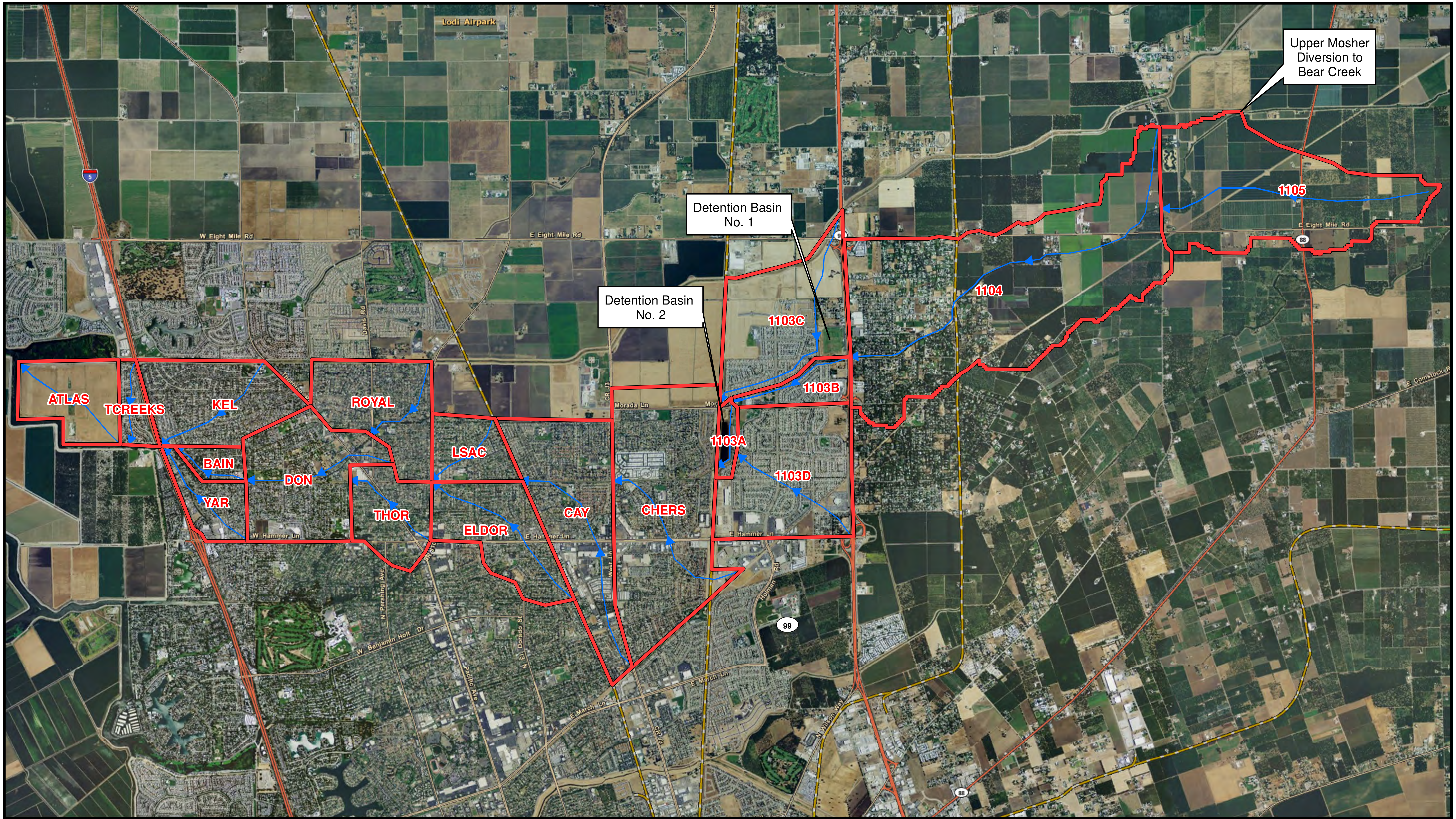
Basin lag times were calculated according to guidelines set forth in the San Joaquin County Hydrology Manual[10]. The following equation was used:

$$Lg = 24n(L \cdot L_C / S^{0.50})^{0.38}$$

where:

Lg	=	Lag time [hours]
n	=	Average basin factor estimated using Figure E-2 in the San Joaquin County Hydrology Manual
L	=	Length of longest watercourse [miles]
L_C	=	Length of longest watercourse measured to the centroid of the basin [miles]
S	=	Overall slope of longest watercourse [feet/mile]

L, L_C, and S were calculated using ArcGIS software. Flowpaths identified for these calculations are shown in Figure 4- 5.



	Subbasin Boundary
✱	Subbasin Centroid
	Subbasin Flowpath

N

0 1,000 2,000 4,000 Feet

1 inch = 4,000 feet

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SAN JOAQUIN AREA FLOOD CONTROL AGENCY

MOSHER SLOUGH SUBBASIN FLOWPATHS

FIGURE

4-5

4.3.5. Channel Routing

The PBI Model utilizes the Muskingum-Cunge routing method to represent attenuation of flood waves within Mosher Slough channels. Routing reach lengths and slopes were measured using ArcGIS software. Manning's n values and channel cross-sections were imported from the 1998 SJAFCA HEC-1 model.

Table 4- 2 provides a summary of routing elements included in the PBI Model.

Table 4- 2. Summary of Mosher Slough model routing elements.

Routing Element	Length [ft]	Slope [ft/ft]	Manning's n Main Channel	Manning's n Overbank	Description From	Description To
R1104	16,260	0.0009	0.035	0.06	1105	1104
R0403	5,760	0.0009	0.035	0.06	1104	1103B/1103C
R3B3A	3,230	0.0015	0.035	0.06	1103B/1103C	1103A
RNC	4,620	0.0011	0.035	0.06	1103A	CHER
RCC	4,130	0.0004	0.035	0.06	CHER	CAY
RCE	3,880	0.0013	0.035	0.06	CAY	ELD
RET	3,700	0.0011	0.035	0.06	ELD	THOR
RTD	4,540	0.0020	0.035	0.06	THOR	DON/RYAL
RLSAC	3,030	0.0010	0.035	0.06	LSAC	RYAL
RRYAL	5,930	0.0022	0.035	0.06	RYAL	DON
RYB	1,740	0.0006	0.035	0.06	DON/RYAL	BAIN
RBK	720	0.0014	0.035	0.06	BAIN	KELLY/YAR
RKT	1,790	0.0006	0.035	0.06	KELLY/YAR	TCREEKS
RTA	8,210	0.0004	0.035	0.06	TCREEKS	ATLAS

Fourteen reaches covering a total of approximately 13 miles of the Mosher Slough stream system are included in the PBI Model.

4.3.6. Loss Rates

As discussed in Section 4.2.1, the 1998 SJAFCA HEC-1 model used the SCS Curve Number method to calculate loss rates. The PBI Model differs from the SJAFCA HEC-1 model in that it uses the initial and constant loss rate method to model subbasin losses.

The Natural Resources Conservation Service (NRCS) has classified all soils into four hydrologic soil groups (A, B, C, and D) according to their infiltration rates[15]:

Table 4- 3. NRCS hydrologic soil groups.

Hydrologic Soil Group	Loss Rate Range [in/hr]	PBI's Assumed Loss Rate[a] [in/hr]
A	> 0.30	0.35
B	0.15 - 0.30	0.2
C	0.05 - 0.15	0.1
D	0.00 - 0.05	0.025

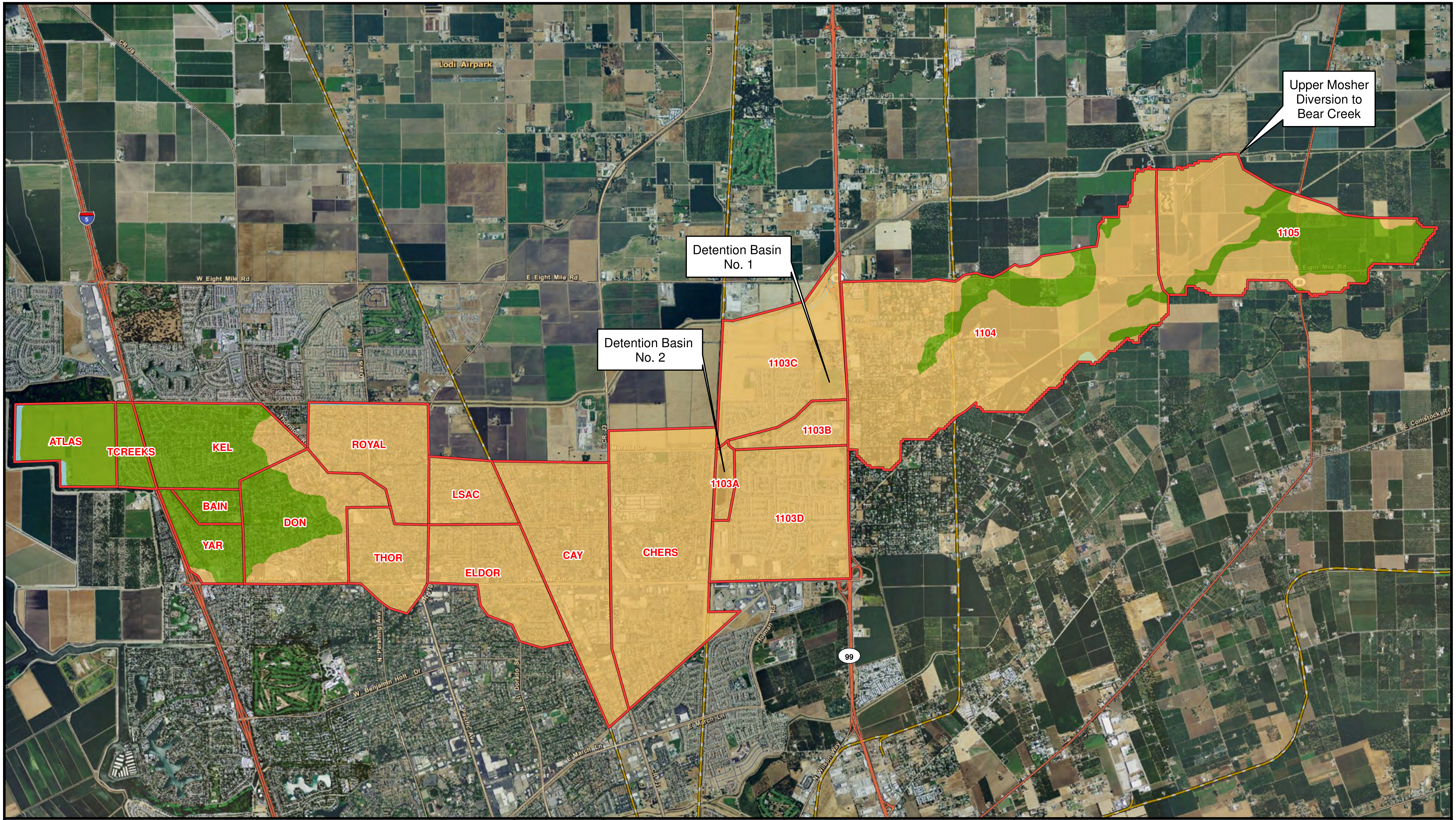
[a]This loss rate value was assigned to each soil group for initial calculations of composite loss rates. The calculated composite loss rates were then adjusted during the calibration process.

A GIS soils layer was obtained from the NRCS[13] and used to determine the proportional coverage of soil groups within Mosher Slough subbasins (Figure 4- 6). A weighted average of loss rates was calculated for each subbasin and adjusted during the calibration process (See Section 4.4). After the calibration adjustment, subbasin loss rates range from 0.02 inches per hour to 0.08 inches per hour as shown in Attachment 4-B.

Initial losses were set at 1.5 inches for pervious areas within all subbasins to account for precipitation that is infiltrated or stored in the watershed before surface runoff begins. This value was selected based on a the Army Corps of Engineers' Comprehensive HEC-HMS study[9] which suggested a range of 1.5 to 2.5 inches for initial losses in the Mosher Slough study area.

4.3.7. Impervious Percentages

Impervious percentages were assigned based on the extent of urbanization within each subbasin. Aerial photos including those contained within 2010 LiDAR datasets[4] covering the Mosher Slough watershed were used to assess existing urbanization. Subbasins were classified into three categories with assumed impervious percentages as shown in Table 4- 4.



Group A
Group B
Group C
Group D
Water/Other

N

0 1,000 2,000 4,000 Feet
1 inch = 4,000 feet

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SAN JOAQUIN AREA FLOOD CONTROL AGENCY

MOSHER SLOUGH SOILS MAP

FIGURE 4-6

The impervious percentages corresponding to each land use type were selected with the guidance of San Joaquin County *Hydrology Manual*[10].

Table 4- 4. Land use types and their corresponding impervious percentages.

Land Use Type	Impervious Percentage
Agricultural	2%
Agricultural with Rural Residential Development	5%
Fully Developed Residential	60%

4.4. MODEL CALIBRATION

The 1998 SJAFCA HEC-1 model documentation does not mention how/if the Mosher Slough model was calibrated. Lower Mosher Slough is largely regulated through pump stations and detention ponds. This flow regulation reduces the importance of model calibration.

Calibration to an observed rainfall/runoff event was considered for the PBI Model, however there was very little concurrent rainfall/runoff data in the Mosher Slough watershed. The available runoff data included stage recordings and did not include a rating curve. Calibration to an observed event would have contained a large amount of uncertainty and therefore was not included in the Mosher Slough analysis.

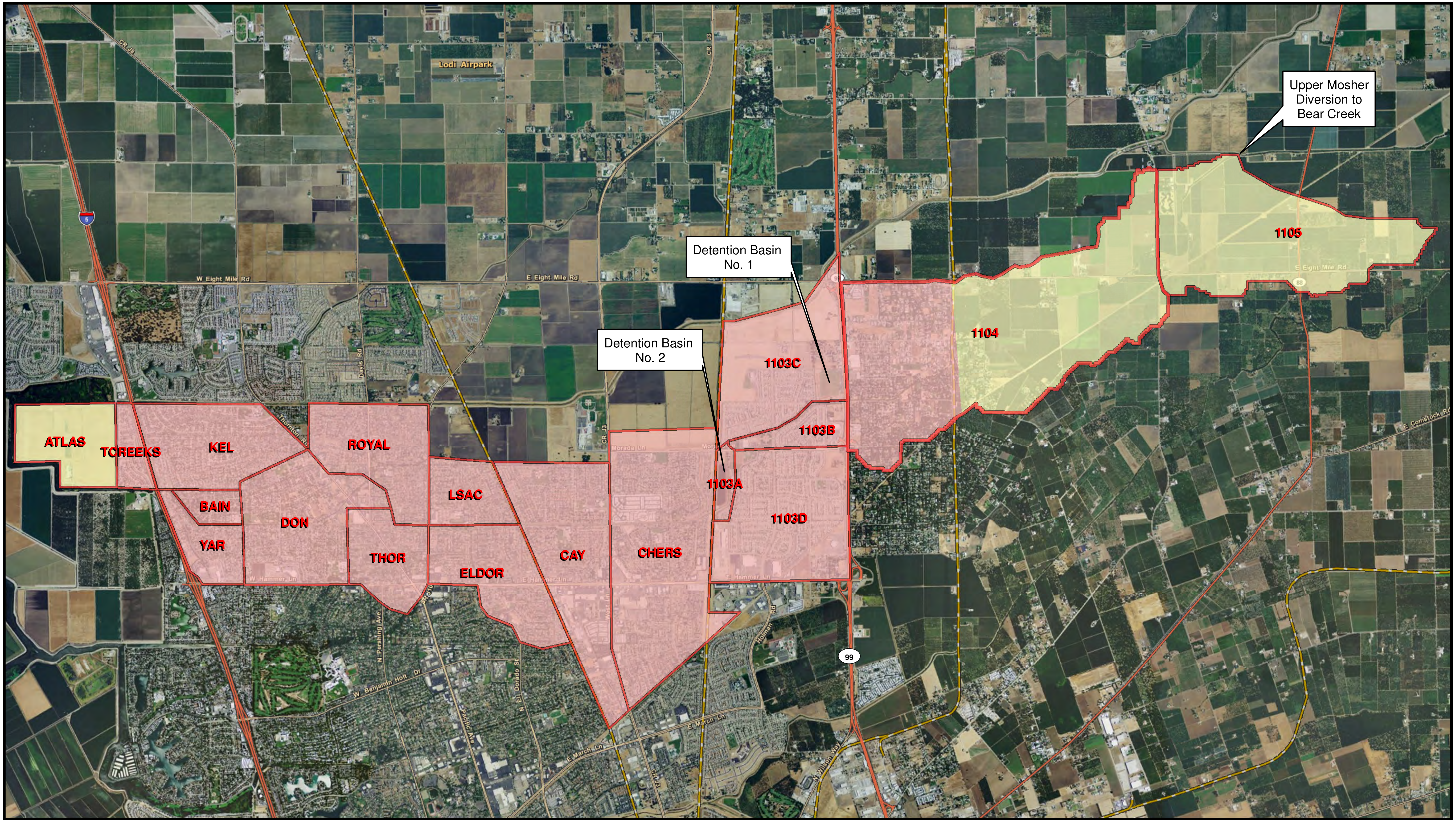
Constant loss rates were adjusted for each subbasin by a factor of 0.80 (Attachment 4- B). The adjustment factor was determined through a HEC-HMS calibration for the neighboring Bear Creek watershed. This watershed has similar characteristics to the Mosher Slough watershed and has more reliable stream flow data. Further details of the Bear Creek model calibration can be found in Section 3.4.

4.5. DEVELOPMENT CONDITIONS

4.5.1. Existing Conditions

An 'Existing Conditions' model run was performed to evaluate peak flows given current (2010) land use and hydrologic conditions within the Mosher Slough watershed. Subbasin S-graphs, 'n' values, and impervious percentages were set according to current land cover conditions using field knowledge supplemented by aerial photos.

As shown in Figure 4- 7, the downstream watershed (west of Highway 99) is considered fully developed and generally consists of residential neighborhoods. Runoff from each of these



Developed Area

Undeveloped Area

N

0 1,000 2,000 4,000 Feet

1 inch = 4,000 feet

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SAN JOAQUIN AREA FLOOD CONTROL AGENCY

EXISTING DEVELOPMENT CONDITIONS FOR MOSHER SLOUGH WATERSHED

FIGURE

4-7

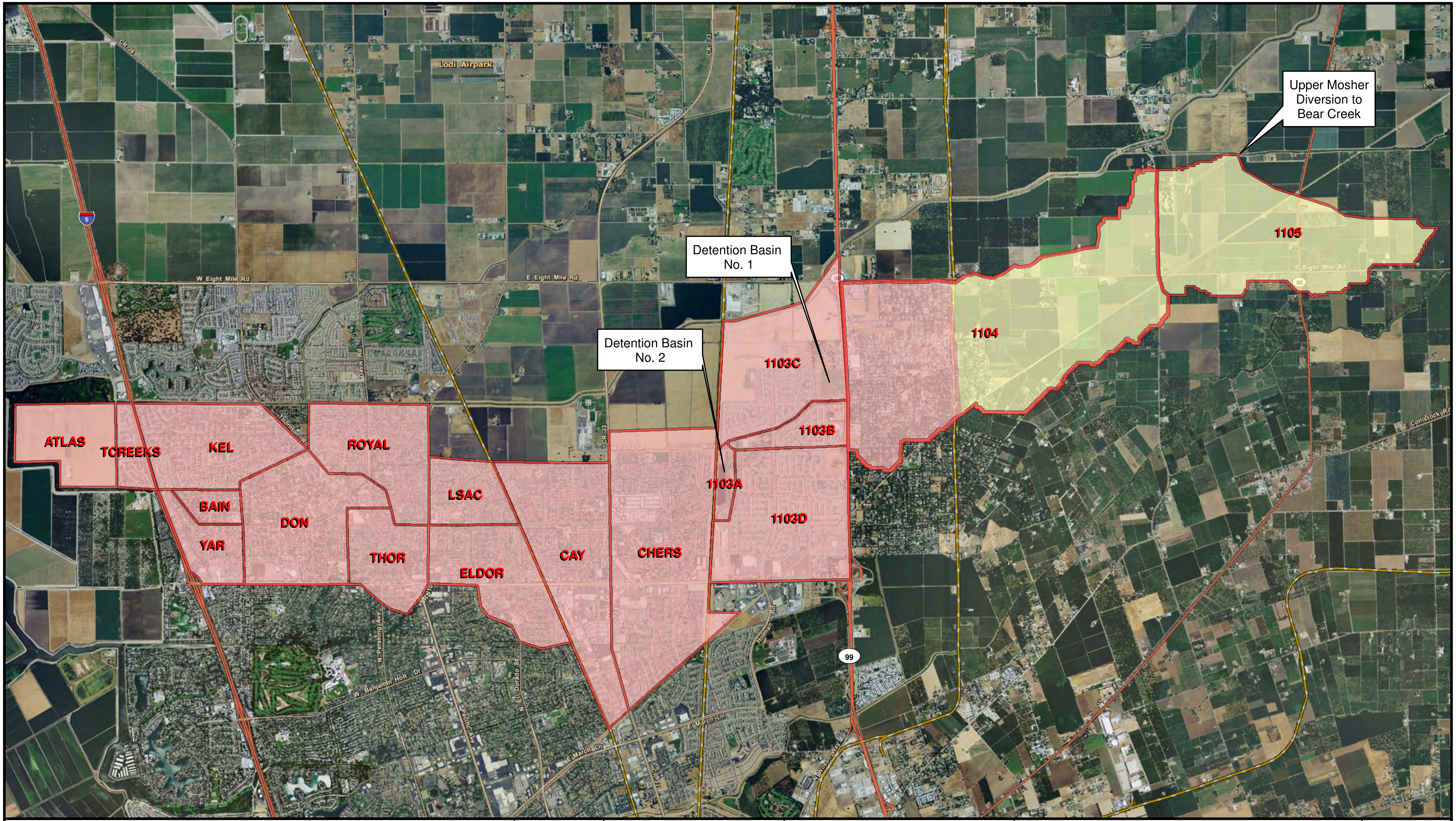
subbasins is routed through pump stations that discharge flows up to their design capacities (see Section 4.3.2) into Mosher Slough. Any subbasin flows exceeding pump station capacities would result in temporary ponding within the subbasin. This ponding would be entirely due to inadequate pump capacities and would be independent of exterior stage conditions in the receiving stream.

The subbasins east of Highway 99 are primarily agricultural lands. Flows from these subbasins are regulated by 'Detention Basin No. 1' as discussed in Sections 4.3.2. A summary table of the subbasin characteristics used for 'Existing Conditions' model runs is provided in Attachment 4-C.

4.5.2. Future-Without-Project Conditions

A 'Future-Without-Project Conditions' model run was performed to evaluate peak flows for future (2070) land use and hydrologic conditions within the Mosher Slough watershed. Land use conditions are based on the City of Stockton 2035 General Plan[12] and the San Joaquin County General Plan[17].

As shown in Figure 4- 8, land use remains largely unchanged from the 'Existing Conditions' model given that most of the watershed was already developed. The only change in land use conditions occurs in the Atlas tract subbasin. This 0.51 square mile area is expected to become developed and is routed through a stormwater pump station into Mosher Slough at a maximum capacity of 120.8 cfs (see Section 3.3.2). A summary table of the subbasin characteristics used for 'Future-Without-Project Conditions' model runs is provided in Attachment 4-D.

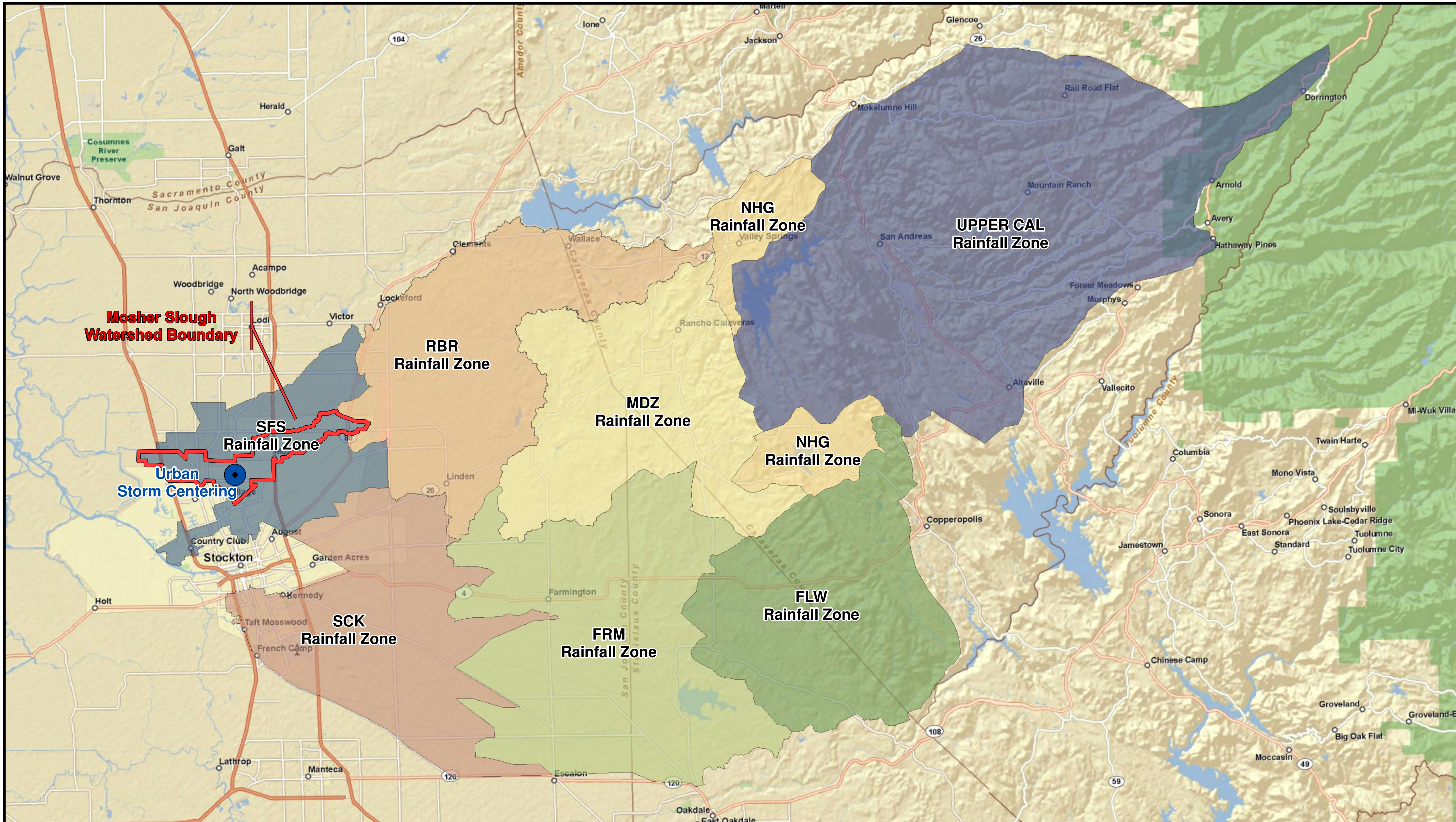


Developed Area Undeveloped Area	N	0 1,000 2,000 4,000 Feet 1 inch = 4,000 feet **AUGUST 20, 2010**	PETERSON . BRUSTAD . INC ENGINEERING . CONSULTING 1180 Iron Point Rd., Suite 260 Folsom, CA 95630 Phone: (916) 608-2212 Fax: (916) 608-2232	**SAN JOAQUIN AREA FLOOD CONTROL AGENCY** **FUTURE DEVELOPMENT CONDITIONS FOR MOSHER SLOUGH WATERSHED**	**FIGURE** **4-8**

4.6. STORM CENTERINGS

Because of the smaller size of the watershed, only one storm centering was analyzed for Mosher Slough (Figure 4- 9). This urban centering was placed directly over the center of the watershed and the 8 AEP storm frequencies were analyzed.

Calculated area reduction factors and resulting area-reduced rainfall depths for each rainfall zone are provided in Attachment 4-E for all frequency-duration combinations.



	N	0 5 Miles 1 : 300,000 JUNE 24, 2011	PETERSON . BRUSTAD . INC ENGINEERING . CONSULTING 1180 Iron Point Rd., Suite 260 Folsom, CA 95630 Phone: (916) 608-2212 Fax: (916) 608-2232	LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY **MOSHER SLOUGH WATERSHED STORM CENTERINGS**	FIGURE **4-9**

4.7. MODEL SIMULATIONS

Mosher Slough production runs include 16 scenarios with unique combinations of development conditions and storm frequencies.

Table 4- 5. Mosher Slough production run scenarios.

Development Conditions	Storm Centerings	AEP Events
Existing Conditions	Urban	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
Future-Without-Project Conditions (2070)	Urban	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500

4.7.1. Summary of Results

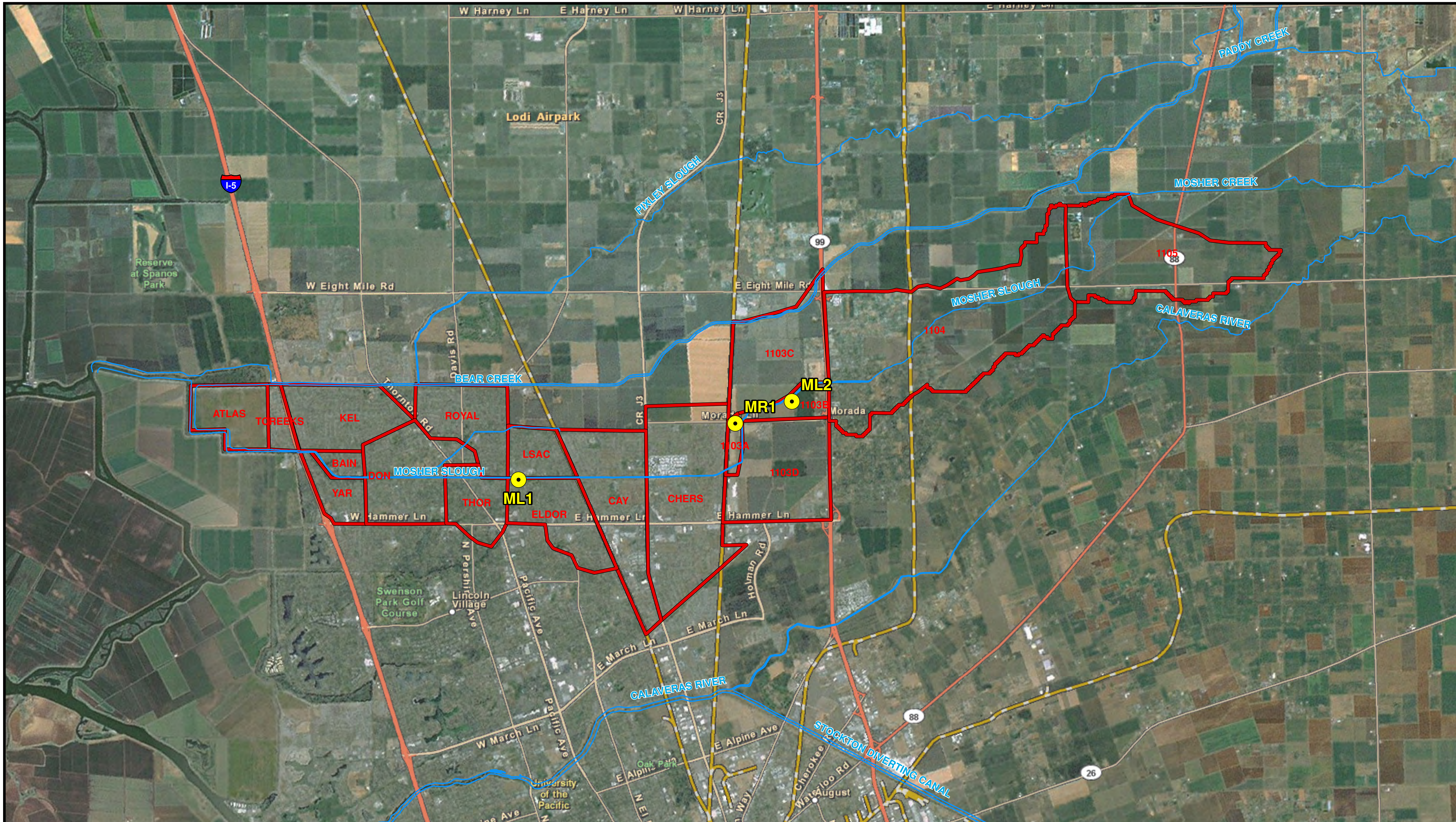
Peak flow results were extracted from HEC-HMS at each LSJRFS index point. Locations of LSJRFS index points within the Mosher Slough watershed are shown in Figure 4- 10. Table 4- 6 and Table 4- 7 summarize peak flows for 'Existing Conditions' runs and for 'Future-Without-Project Conditions' runs, respectively.

4.7.2. Uncertainty Parameters

For the purposes of the LSJRFS, uncertainty parameters for each flow-frequency dataset can be estimated within HEC-FDA during the project's economic analysis. HEC-FDA defines uncertainty in terms of confidence intervals or standard deviations given inputs of flow-frequency data (provided in Table 4- 6 and Table 4- 7) and an equivalent record length.

The equivalent record length is an estimate of the overall "worth" or "quality" of the flow-frequency function, expressed as the number of years-of-record[19]. For probability functions derived at ungaged locations using model or other data, the equivalent record length is based on a judgment of the quality of that model or data. EM 1110-2-1619[20] provides guidelines for assigning equivalent record lengths and estimates that a rainfall-runoff model calibrated to an observed event at a short-interval runoff gage has an equivalent record length of 20-30 years.

The Mosher Slough model wasn't calibrated to an observed event, however, because stream flows are largely dependent on pumped flows, the degree of uncertainty is judged to be equivalent to a calibrated model.



- LSJRFS Index Point
- Subshed Boundary

1 inch = 1 mile

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LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

MOSHER SLOUGH WATERSHED INDEX POINTS

FIGURE 4-10

Table 4-6. Peak Flow Results for Mosher Slough - Existing Conditions [cfs]

LSJRFS Index Point ID	Description	Urban Storm Centering							
		1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP
--	Mosher Slough at Hwy 99	170	240	290	360	420	470	530	600
ML2	Mosher Slough d/s of Detention Basin #1	170	230	230	230	230	230	230	510
--	Mosher Slough at SPRR (d/s of La Morada)	180	240	270	320	320	320	320	590
--	Mosher Slough at UPRR	390	530	570	640	730	770	780	840
ML1	Mosher Slough at El Dorado St.	440	620	690	800	890	940	960	970
--	Mosher Slough at Thornton Ave.	450	590	690	790	890	950	980	1,000
--	Mosher Slough at Don Ave	630	810	930	1,030	1,160	1,230	1,270	1,290
--	Mosher Slough at I-5	690	860	1,040	1,250	1,360	1,420	1,500	1,540
--	Mosher Slough u/s of Bear Creek Confluence	570	750	890	1,050	1,160	1,260	1,390	1,450

Table 4-7. Peak Flow Results for Mosher Slough - Future Conditions [cfs]

LSJRFS Index Point ID	Description	Urban Storm Centering							
		1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP
--	Mosher Slough at Hwy 99	170	240	290	360	420	470	530	600
ML2	Mosher Slough d/s of Detention Basin #1	170	230	230	230	230	230	230	510
--	Mosher Slough at SPRR (d/s of La Morada)	180	240	270	320	320	320	320	590
--	Mosher Slough at UPRR	390	530	570	640	730	770	780	840
ML1	Mosher Slough at El Dorado St.	440	620	690	800	890	940	960	970
--	Mosher Slough at Thornton Ave.	450	590	690	790	890	950	980	1,000
--	Mosher Slough at Don Ave	630	810	930	1,030	1,160	1,230	1,270	1,290
--	Mosher Slough at I-5	690	860	1,040	1,250	1,360	1,420	1,500	1,540
--	Mosher Slough u/s of Bear Creek Confluence	590	760	910	1,070	1,190	1,290	1,400	1,480

5.0 CALAVERAS RIVER HEC-HMS MODELING

5.1. GENERAL

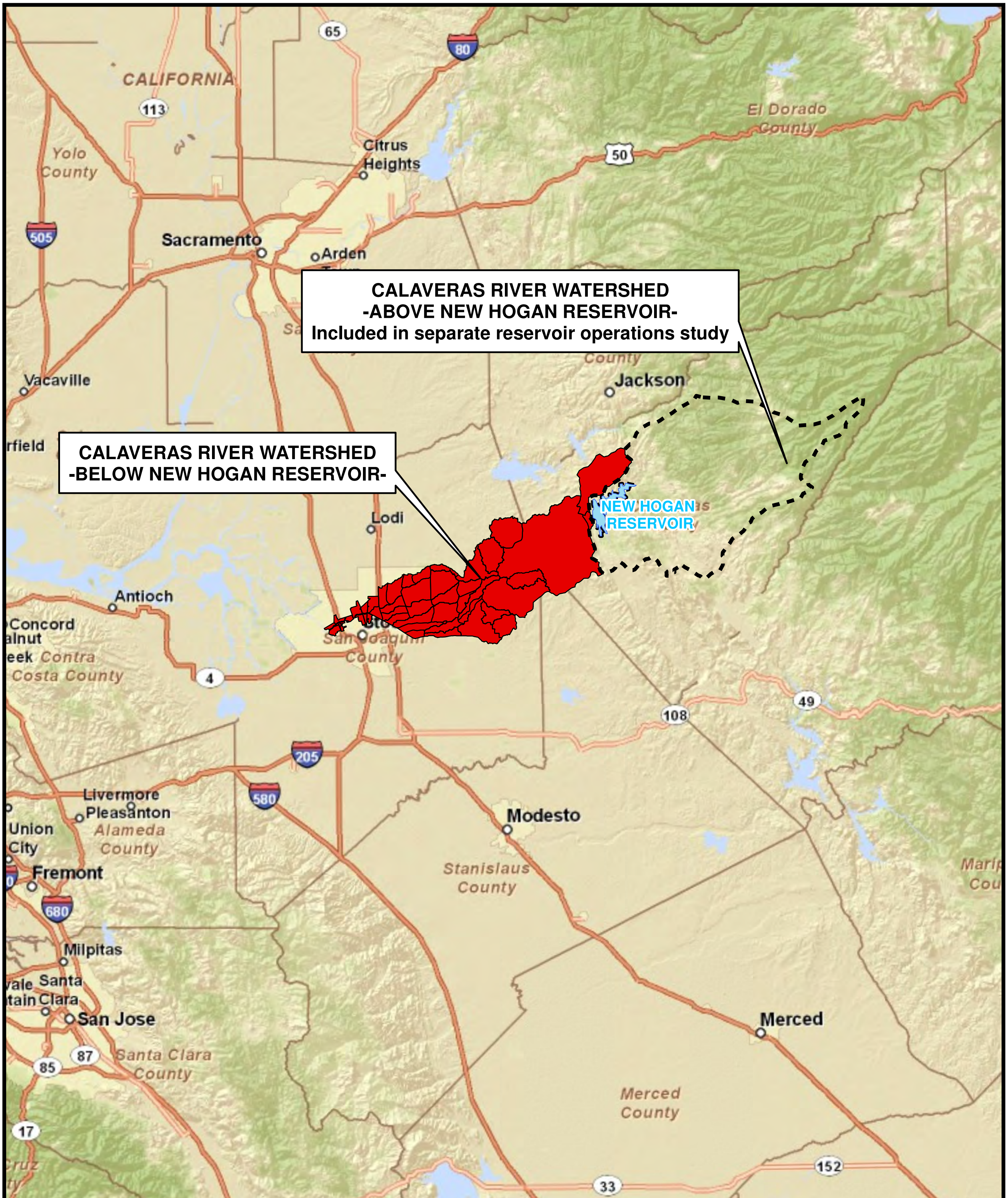
5.1.1. Location

The Calaveras River watershed is located near the city of Stockton in San Joaquin County, California (Figure 5- 1). The watershed runs east from the city of Stockton into the Sierra Nevada foothills in Calaveras County. The Calaveras River watershed can be split into two sections: above New Hogan Dam and below New Hogan Dam. This document focuses on the section of the Calaveras River below the dam whereas the section above the dam is part of a separate reservoir operations study[21].

The watershed includes a total area of 597 square miles with 352 square miles of this tributary area flowing into New Hogan Reservoir. The watershed discussed in this TM (below New Hogan Reservoir) includes the remaining 245 square miles and achieves maximum elevations of 1,500 feet. It then descends through moderate slopes to the lower portion of the watershed which lies at sea-level. Flow in the stream system is largely affected by releases from New Hogan Reservoir. The entire watershed is low enough in elevation to be rainfall dominant. The HEC-HMS model described in this memorandum includes the Calaveras River, Cosgrove Creek, Mormon Slough, Potter Creek, and the Stockton Diverting Canal systems and discharges to the San Joaquin River to the west of Interstate-5.

5.1.2. Topography

The HEC-HMS model utilized for this study is titled the PBI Calaveras River Model (PBI Model) which is georeferenced to the NAD 1983 State Plane California Coordinate System Zone III (U.S. Survey Feet). Vertical elevations are reported in the NAVD 1988 datum. Topography used for model development included United States Geological Survey (USGS) 30-meter Digital Elevation Models (DEMs)[3]. Where available, Department of Water Resources (DWR) LiDAR data[4] was also used to confirm subbasin boundaries.



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SAN JOAQUIN AREA FLOOD CONTROL AGENCY

FIGURE 5-1

VICINITY MAP OF THE CALAVERAS RIVER STUDY AREA

5.2. MODEL DEVELOPMENT

The PBI model was developed using HEC-HMS version 3.4[5] and HEC-GeoHMS version 4.2[6]. A summary of the tasks performed are listed below:

1. The 1998 SJAFCA HEC-1 model was imported into HEC-HMS (See Section 5.2.2).

2. Subbasin boundaries were updated using HEC-GeoHMS and United States Geological Survey (USGS) Digital Elevation Models (DEMs)[3] (See Section 5.3.1).

3. Lower Calaveras River subbasins (below the confluence with the Diverting Canal) were added to the PBI Model (See Section 5.3.1).

4. Pump stations were coded into the PBI Model based on design pumping rates provided by the City of Stockton[7] (See Section 5.3.2).

5. New Hogan Reservoir outflows were determined through a separate reservoir operations study (See Section 5.3.3)

6. Diversions and channel routing parameters were coded into the PBI Model (See Sections 5.3.4 and 5.3.6, respectively).

7. S-graphs and lag times were assigned to each subbasin (See Section 5.3.5).

8. Loss rates and impervious percentages were coded into the PBI Model (See Section 5.3.7 and Section 5.3.8).

9. The PBI Model was calibrated using historical rainfall and runoff data (See Section 5.4).

10. The PBI Model was set up to simulate both ‘Existing’ (see Section 5.5.1) and ‘Future-Without-Project’ (see Section 5.1.1) scenario runs.

5.2.1. SJAFCA HEC-1 Model

The PBI Model is a conversion and update of the HEC-1 model developed for SJAFCA by HDR Engineering, Inc. in 1998[8].

The 1998 SJAFCA HEC-1 model utilized the San Joaquin County LA preprocessor to convert S-graphs to unit hydrographs for each subbasin. Three types of S-graphs were obtained from the San Joaquin County Hydrology Manual[10] and used based on the surface condition classification of the subbasin: Foothill, Valley Undeveloped, and Valley

Developed. Lag times were calculated by HDR using basin 'n', length of subbasin flow, flow length from the centroid, and slope of the basin.

The SJAFCA HEC-1 model used the SCS curve number method to account for subbasin losses. Curve numbers typically ranged from 70 to 84 depending on soil type and cover.

Calibration of the SJAFCA HEC-1 model included calculating the flow per square mile for the 1/100 AEP event at the Duck Creek near Farmington gage and comparing it to the modeled flow per square mile coming from the foothill portions of Potter Creek. HDR made adjustments to basin 'n' values such that the 1/100 AEP rainfall event would produce the 1/100 AEP streamflow event.

5.2.2. Conversion from HEC-1 to HEC-HMS

The 1998 SJAFCA HEC-1 model was imported into HEC-HMS as the fundamental basis for the PBI Model. Parameters from the HEC-1 model are listed in Attachment 5-A.

Certain features in the HEC-1 software are not supported in HEC-HMS and therefore were not properly transferred during the import process. Pump station data and meteorological data from the SJAFCA HEC-1 model were manually coded into the PBI Model so as to conform to HEC-HMS formatting.

In addition, there are computational differences between the HEC-1 and the HEC-HMS software. One such difference involves the Muskingum-Cunge stream segment routing technique used for the PBI model. In HEC-HMS, channel properties are computed based on the physical characteristics of that channel, whereas in HEC-1 the properties are computed with formulas based on a kinematic wave assumption[5]. This causes minor differences in the flows that are transferred through the routing parameters. HEC-HMS results are preferred because of the refined computational techniques that have been implemented.

Once the conversion from HEC-1 to HEC-HMS was completed successfully, the HEC-HMS model was modified to run with updated features.

5.3. MODEL FEATURES

The 1998 SJAFCA HEC-1 model was converted and modified for this study to form the PBI HEC-HMS Model. The PBI Model components are described in the following sections.

5.3.1. Subbasins

Subbasin boundaries used in the 1998 SJAFCA HEC-1 model were cross-checked with USGS 30-meter DEM datasets[3] and modified where appropriate. Subbasin boundaries were delineated using the ArcHydro and HEC-GeoHMS[6] extensions within the ArcGIS software package. These tools utilize geospatial data to interpret drainage patterns and delineate watershed boundaries accordingly. Subbasin outlet points were set similar to the locations utilized in the SJAFCA HEC-1 model. Where available, DWR LiDAR[4] data was used to confirm subbasin boundaries.

The 1998 SJAFCA HEC-1 model boundary was extended along the Lower Calaveras River (below the confluence with the Diverting Canal) by adding 12 subbasins. These subbasin boundaries were based on the existing storm drain system and the City of Stockton's *Conceptual Storm Drain Master Plan*[11]. The Calaveras River subbasins included in the PBI Model are shown in Figure 5- 2.

Subbasin 'C80' from the SJAFCA HEC-1 Model was renamed to 'HOLM' as it corresponds to the Holman stormwater pump station's drainage area.

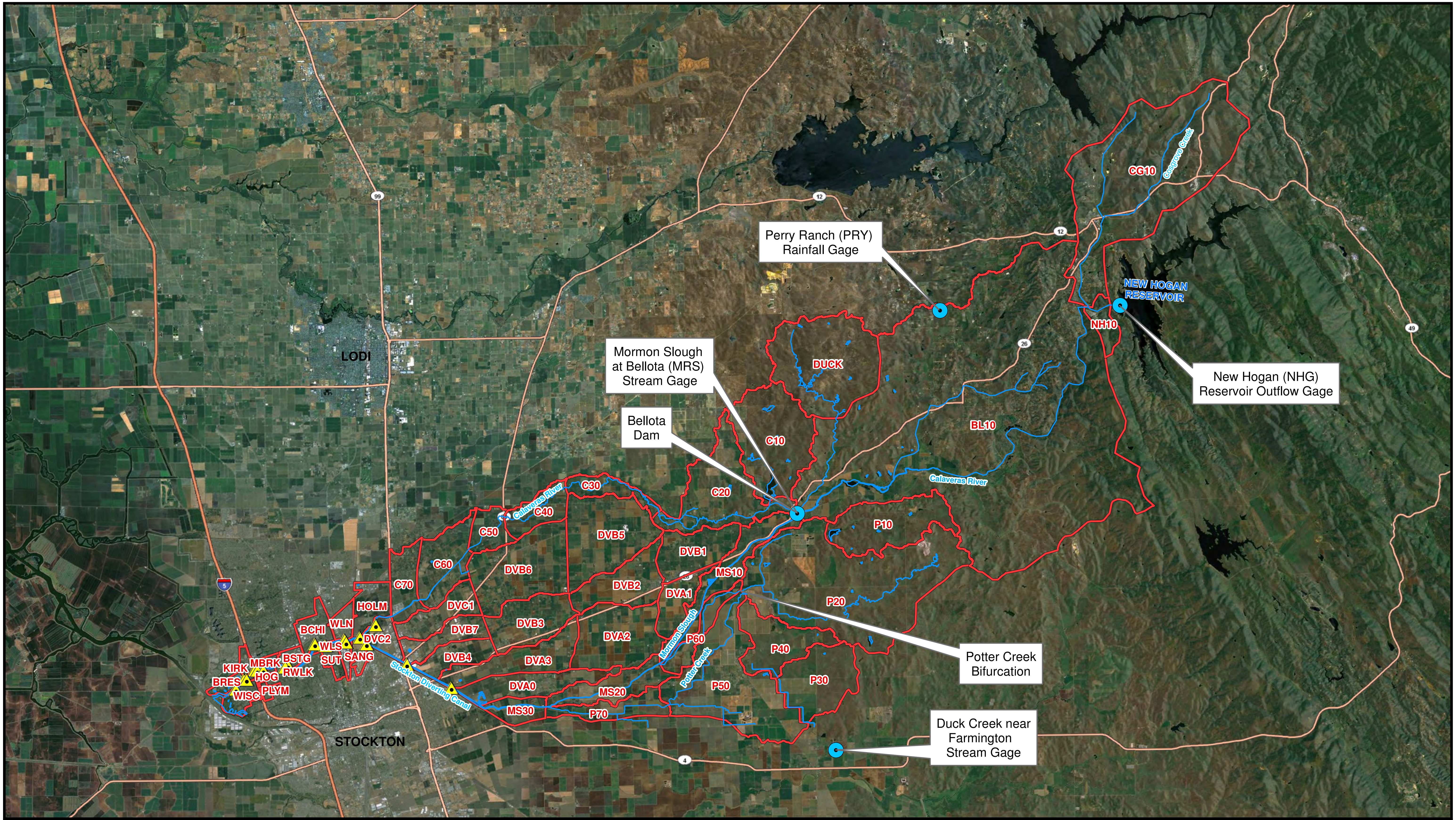
The PBI Model contains a total of 48 subbasins with drainage areas ranging from 0.02 square miles to 72.63 square miles and a total watershed area of approximately 245 square miles.

For subbasins that are on the outside of a levee which do not have pump stations, runoff is coded to enter the main channel at road crossings where there are through-levee culverts. The assumption is made that the culvert headgates will remain open and allow outside flow to enter the main channel. This assumption was made to remain conservative and to account for the potential replacement of culverts by pump stations in the future.

The GIS horizontal coordinates for each subbasin were used to georeference model elements within the PBI HEC-HMS Model. The subbasin GIS shapefile was inserted into the PBI Model as a background map.

5.3.2. Pump Stations

Pump stations were included in the PBI Model to represent storm drain conveyance from developed subbasins to the main channels. There are sixteen (16) pump stations included in the PBI model. Pumps along the Diverting Canal were imported directly from the 1998 SJAFCA HEC-1 Model. Pump stations along the Lower Calaveras River (below the confluence with the Diverting Canal) include multiple pumps with capacities assigned based on City of Stockton records[7]. All pumps are set to discharge over the top of the levees and into the receiving channel above the highest stage expected. The exterior and interior areas at the pump stations are independent from one another.



Subbasin Boundary
Existing Pump Station

N

0 1.5 3 Miles
1 inch = 3 miles

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SAN JOAQUIN AREA FLOOD CONTROL AGENCY

CALAVERAS RIVER
HEC-HMS SUBBASINS

FIGURE
5-2

Table 5- 1 provides a summary of pump stations included in the PBI Model.

Table 5- 1. Summary of Calaveras River pump stations.

Pump Station Name	Contributing Subbasin(s)	Pump Station Capacity [cfs]	Pump Station Notes
Brookside Estates South	BRES	39.0	3 @ 12.0 cfs 1 @ 2.9 cfs
Wisconsin	WISC	21.7	1 @ 10.2 cfs 1 @ 11.5 cfs
March-Brookside	MBRK	121.3	1 @ 7.7 cfs 3 @ 37.9 cfs
Kirk	KIRK	14.5	1 @ 14.5 cfs
Plymouth	PLYM	6.0	1 @ 6.0 cfs
Hogue-Tyler	HGTY	6.2	1 @ 6.2 cfs
Riverwalk	RWLK	10.5	1 @ 10.5 cfs
Brookside-Stagg	BSTG	132.7	1 @ 7.9 cfs 2 @ 62.4 cfs
Bianchi	BCHI	176.9	2 @ 1.6 cfs 1 @ 22.3 cfs 2 @ 64.6 cfs 1 @ 13.4 cfs 1 @ 8.9 cfs
Sutter	SUT	54.1	1 @ 11.1 cfs 1 @ 20.7 cfs 1 @ 22.3 cfs
West Lane - North	WLN	254.0	1 @ 8.9 cfs 5 @ 49.0 cfs
West Lane - South	WLS	47.5	1 @ 36.3 cfs 1@ 11.1 cfs
Sanguinetti	SANG	92.2	1 @ 9.8 cfs 2 @ 41.2 cfs
Holman	HOLM	140.1	2 @ 34.5 cfs 1 @ 2.0 cfs 2 @ 34.5 cfs
Diverting Canal & Route 26 (P1)	DIVA0, DIVA1, DIVA2, DIVA3	16.0	1 @ 16.0 cfs
Diverting Canal & HWY 99 (P2)	DIVB1, DIVB2, DIVB3, DIVB4, DIVB5, DIVB6, DIVB7 + Excess from P1	100.0	1 @ 100 cfs

There is an additional pump station located along the Diverting Canal at its confluence with the Calaveras River. The coding of this model element (P-OUT) includes the combined flow from a small pump station and two 6'x 6' reinforced concrete box culverts that relieve ponding behind the Diverting Canal.

5.3.3. New Hogan Reservoir

David Ford Consulting Engineers (DFCE) completed a separate reservoir operations analysis for New Hogan Reservoir as part of the LSJRFS[21]. This analysis was later amended by USACE as documented in their *Draft Memorandum for Record: Lower San Joaquin River Feasibility Study, Bellota and Farmington Regulated Flow Hydrographs* (07 FEB 2012)[22]. One of the final deliverables from this study was regulated hydrographs at the Bellota control point for each of the 8 LSJRFS AEP storm events. These hydrographs include all flows coming out of New Hogan Dam along with all local flows upstream of Bellota. These regulated flow hydrographs were coded into the PBI HEC-HMS model as time-series discharge gages and supersede all HEC-HMS inflow that comes from above Bellota.

The Ford report and the USACE amendment should be referenced for any details regarding the reservoir operations study.

The following table is based on the information in the USACE amendment and shows the flow-frequency relationship for modeled flows at the Bellota control point.

Table 5- 2. Flow-frequency at Bellota Control Point

Regulated Peak Flow values and associated volumes: Mormon Slough at Bellota					
Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes1 (as average flow for given duration) 1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)
0.5	3,515	2,491	2,400	2,144	1,527
0.2	9,515	7,702	7,164	6,053	4,562
0.1	9,388	8,527	7,560	6,102	5,345
0.04	10,319	9,307	9,206	7,943	5,485
0.02	12,500	10,300	9,900	9,400	7,800
0.01	12,500	11,400	11,300	10,900	10,100
0.005	12,500	12,400	12,200	12,000	11,300
0.002	16,000	13,500	13,100	13,000	12,500

Revised to reflect graphical fit of observed data from Jan1988 to Sep2010 for the 0.5 to the 0.04 AEP; the graphically fit data was further refined to fit the local flow frequency data by PBI. The 0.02 to 0.002 AEP events are from the revised flow transform and regulated flow-freq curve. The volumes were computed from the regulated peak to , volume transforms in the Ford report and were warped to mesh with the graphically derived peak and volume flow for the 0.5 to 0.04 AEP events.

5.3.4. Diversions

Diversions in HEC-HMS are coded to simulate either manmade diversions or topographic flow splits. Several diversions were imported from the 1998 SJAFCA HEC-1 Model and included in the PBI Model.

Calaveras River flows are completely diverted to Mormon Slough at Bellota (see Figure 5-2). This makes subbasin C10 the initial tributary basin of the Upper Calaveras River downstream of the Bellota diversion.

The Upper Calaveras River includes two topographic diversions located downstream of Jack Tone Road. The berms along this segment of the river prevent most subbasin runoff from entering the main channel. The diversions are used to route a portion of subbasin flow through small culvert inlets at Jack Tone Road and at Highway 88. The remainder of subbasin flow is routed overland to subsequent subbasins and ultimately enters the main channel at Highway 99.

Potter Creek splits into two main branches at a location downstream of Gilmore Reservoir. A diversion element is included in the PBI Model to represent this bifurcation. Cross-sections of the two stream branches were used to determine the proper split of flow at this location.

Two diversion elements are used to represent pump stations located along the Diverting Canal. These relatively small pump stations help to relieve ponded flooding against the east levee. Any runoff coming from upstream subbasins that exceeds pump capacities is diverted overland to downslope subbasins and pump stations.

5.3.5. S-graphs and Lag Times

As discussed in Section 5.2.1, the 1998 SJAFCA HEC-1 model utilized the San Joaquin County LA preprocessor for converting S-graphs and to unit hydrographs. The PBI Model assigns Foothill, Valley Undeveloped, and Valley Developed S-graphs directly into HEC-HMS for each subbasin based on its location. S-graph data points were obtained from the San Joaquin County Hydrology Manual[10]. The S-graphs were developed based on rainfall-runoff data from Southern California catchments considered to be hydrologically similar to the local catchments. The following figures show the time versus discharge relationship for each S-graph.

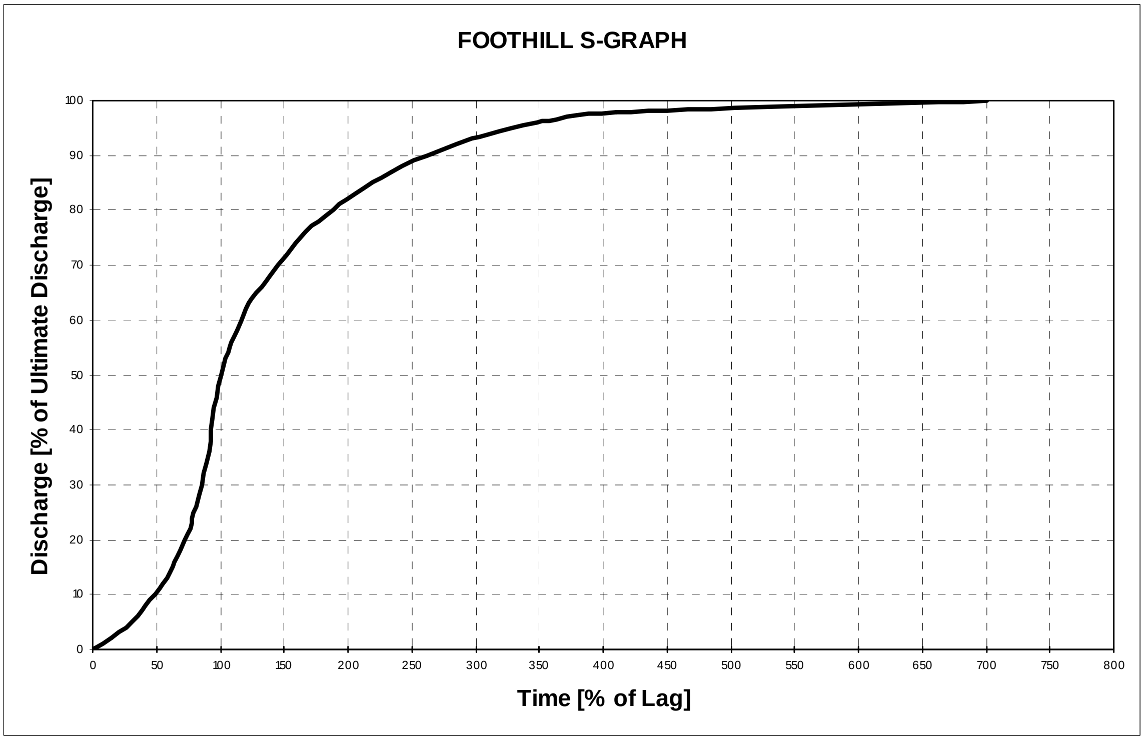


Figure 5- 3. San Joaquin County Foothill S-graph

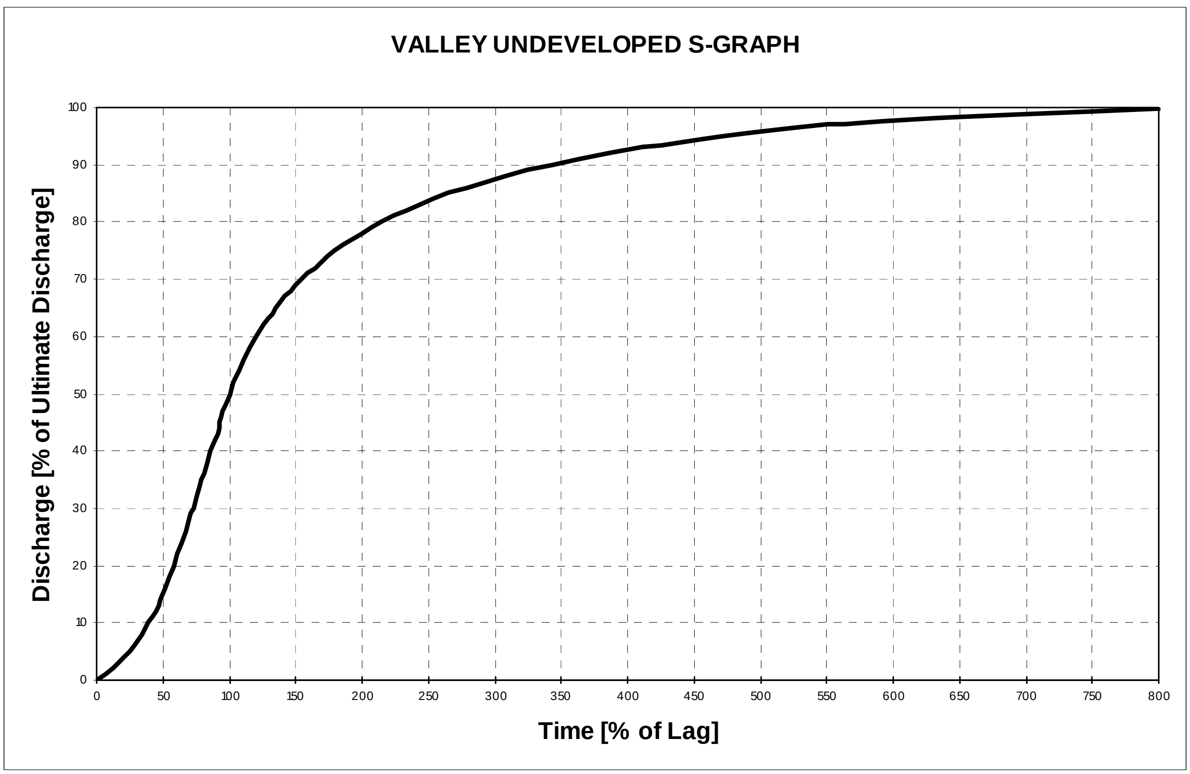


Figure 5- 4. San Joaquin County Valley Undeveloped S-graph

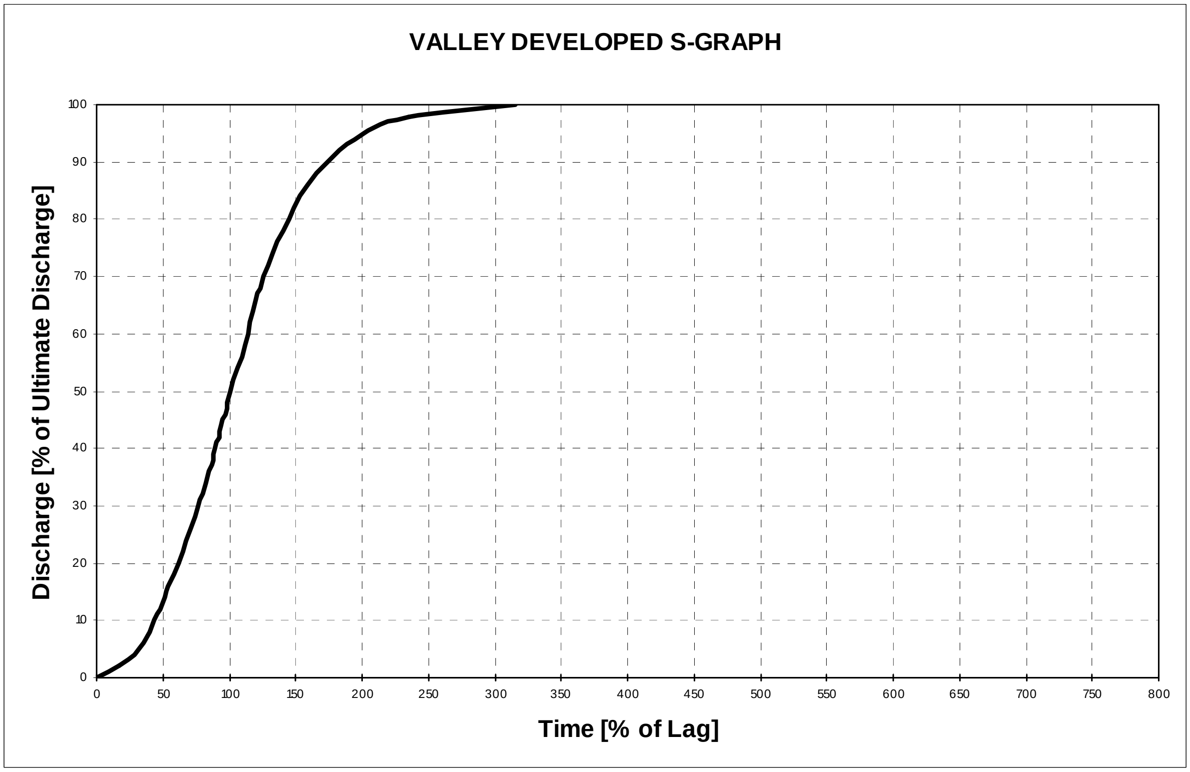


Figure 5- 5. San Joaquin County Valley Developed S-graph

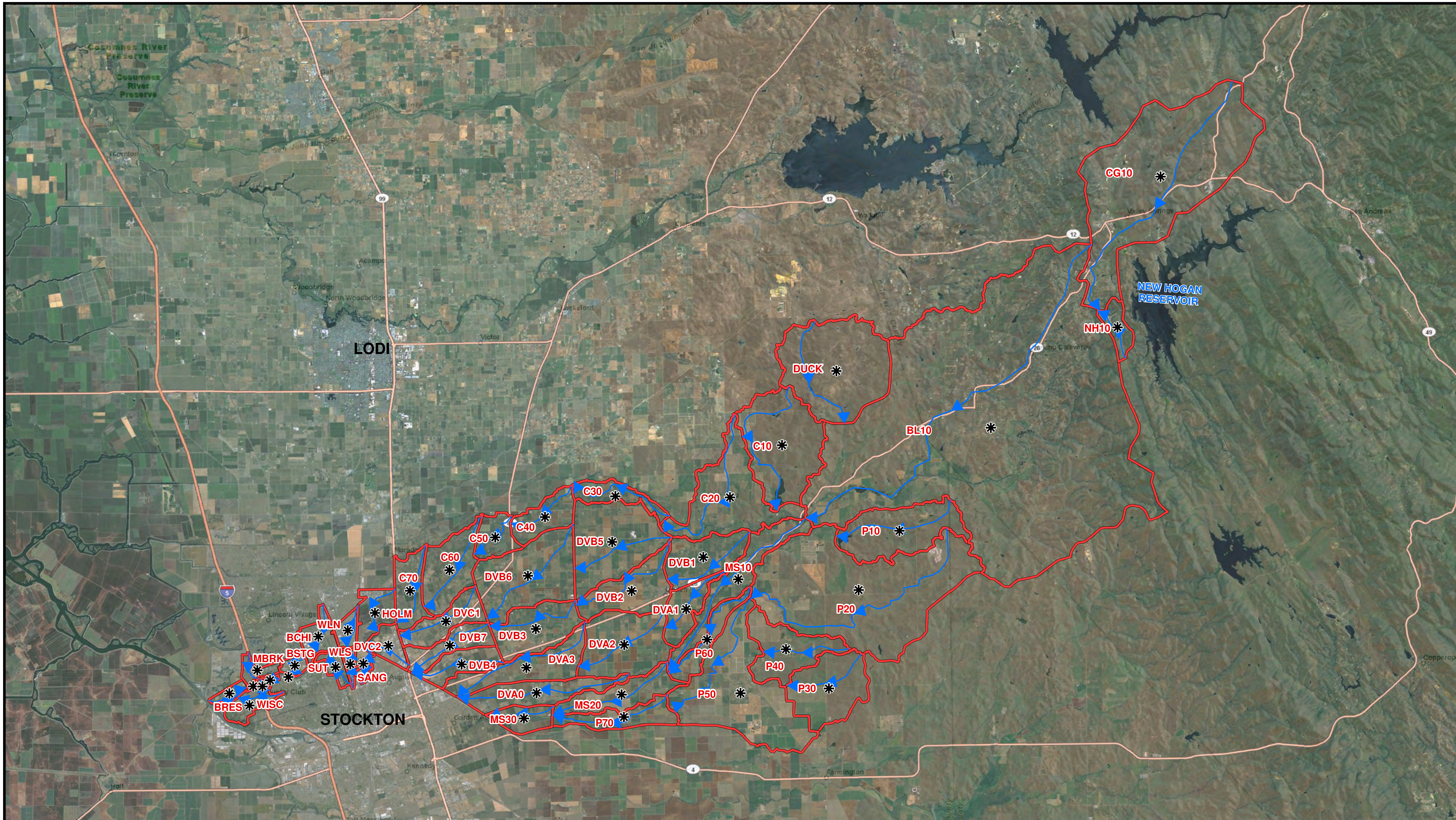
Basin lag times were calculated according to guidelines set forth in the San Joaquin County Hydrology Manual[10]. The following equation was used:

$$Lg = 24n(L \cdot L_C/S^{0.50})^{0.38}$$

where:

Lg	=	Lag time [hours]
n	=	Average basin factor estimated using Figure E-2 in the San Joaquin County Hydrology Manual
L	=	Length of longest watercourse [miles]
L_C	=	Length of longest watercourse measured to the centroid of the basin [miles]
S	=	Overall slope of longest watercourse [feet/mile]

L, L_C, and S were calculated using ArcGIS software. Flowpaths identified for these calculations are shown in Figure 5- 6. S-graph assignments and lag time calculations for each subbasin are provided in Attachment 5-B.



Subbasin Boundary

Subbasin Centroid

Subbasin Flowpath

N

0 1.5 3 Miles

1 inch = 3 miles

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CALAVERAS RIVER SUBBASIN FLOWPATHS

FIGURE

5-6

5.3.6. Channel Routing

The PBI Model utilizes the Muskingum-Cunge routing method to represent attenuation of flood waves within Calaveras channels. Routing reach lengths and slopes were measured using ArcGIS software. Manning's n values and channel cross-sections were imported from the 1998 SJAFCA HEC-1 model[8].

Table 5- 3 provides a summary of routing elements included in the PBI Model.

Table 5- 3. Summary of Calaveras River model routing elements.

Routing Element	Length [ft]	Slope [ft/ft]	Manning's n: Main Channel	Manning's n: Left Overbank	Manning's n: Right Overbank	Description: From	Description: To
RBL	77,630	0.0049	0.035	0.05	0.05	CG10-NH10	BL10
RDUCK	19,500	0.0022	0.035	0.05	0.05	DUCK	BL10
R1010	36,830	0.0010	0.035	0.08	0.05	BL10-P10	MS10-P60
R2060	21,680	0.0009	0.04	0.08	0.08	P20	P60
R1020M	22,100	0.0014	0.035	0.06	0.06	MS10-P60	MS20
R1020P	22,950	0.0017	0.04	0.08	0.08	P10	P20
R2050	28,380	0.0009	0.04	0.08	0.08	P20	P30-P40-P50
R5070	22,350	0.0011	0.045	0.05	0.05	P50	P70
R2030M	11,280	0.0011	0.04	0.08	0.08	MS20-P70	MS30
R7080	7,040	0.0017	0.04	0.08	0.08	MS30	DIVA0-DIVA3
R8090	8,410	0.0004	0.04	0.08	0.08	DIVA0-DIVA3	DIVB4-DIVB7
R9092	8,370	0.0005	0.04	0.08	0.08	DIVB4-DIVB7	SANG
R92	1,590	0.0006	0.04	0.08	0.08	SANG	HOLM-DIVC2
R1020	22,460	0.0016	0.055	0.08	0.06	C10	C20
R2030	19,630	0.0013	0.055	0.08	0.06	C20	C30
R3040	13,110	0.0015	0.045	0.07	0.075	C30	C40
RJDIV	11,000	0.0011	0.050	0.08	0.080	C30	C40
RDV40	9,000	0.0009	0.050	0.08	0.080	C40	C50
RSRES	18,000	0.0006	0.050	0.08	0.080	C50	C60
R60	5,000	0.0014	0.050	0.08	0.080	C60	C70
R4070	28,260	0.0008	0.05	0.08	0.075	C40	C70
R70	4,610	0.0011	0.05	0.08	0.08	C70	HOLM
R80	3,810	0.0013	0.05	0.08	0.08	HOLM	DIVC2
R100	2,720	0.0004	0.04	0.08	0.08	HOLM-DVC2	WLN-WLS
R110	4,470	0.0009	0.04	0.08	0.08	WLN-WLS	BCHI-SUT
R120	8,170	0.0009	0.04	0.08	0.08	BCHI-SUT	BSTG-RWLK
R130	4,810	0.0010	0.04	0.08	0.08	BSTG-RWLK	MBRK-HGTY-PLYM
R140	3,280	0.0006	0.04	0.08	0.08	MBRK-HGTY-PLYM	BRES-KIRK
R150	2,280	0.0004	0.04	0.08	0.08	BRES-KIRK	WISC
R160	4,560	0.0004	0.04	0.08	0.08	WISC	OUTLET

Thirty reaches covering approximately 85 miles of the Calaveras River and Mormon Slough stream systems are included in the PBI Model.

5.3.7. Loss Rates

As discussed in Section 5.2.1, the SJAFCA HEC-1 model used the SCS Curve Number method to calculate loss rates. The PBI Model differs from the 1998 SJAFCA HEC-1 model in that it uses the initial and constant loss rate method to model subbasin losses.

The Natural Resources Conservation Service (NRCS) has classified all soils into four hydrologic soil groups (A, B, C, and D) according to their infiltration rates[15]:

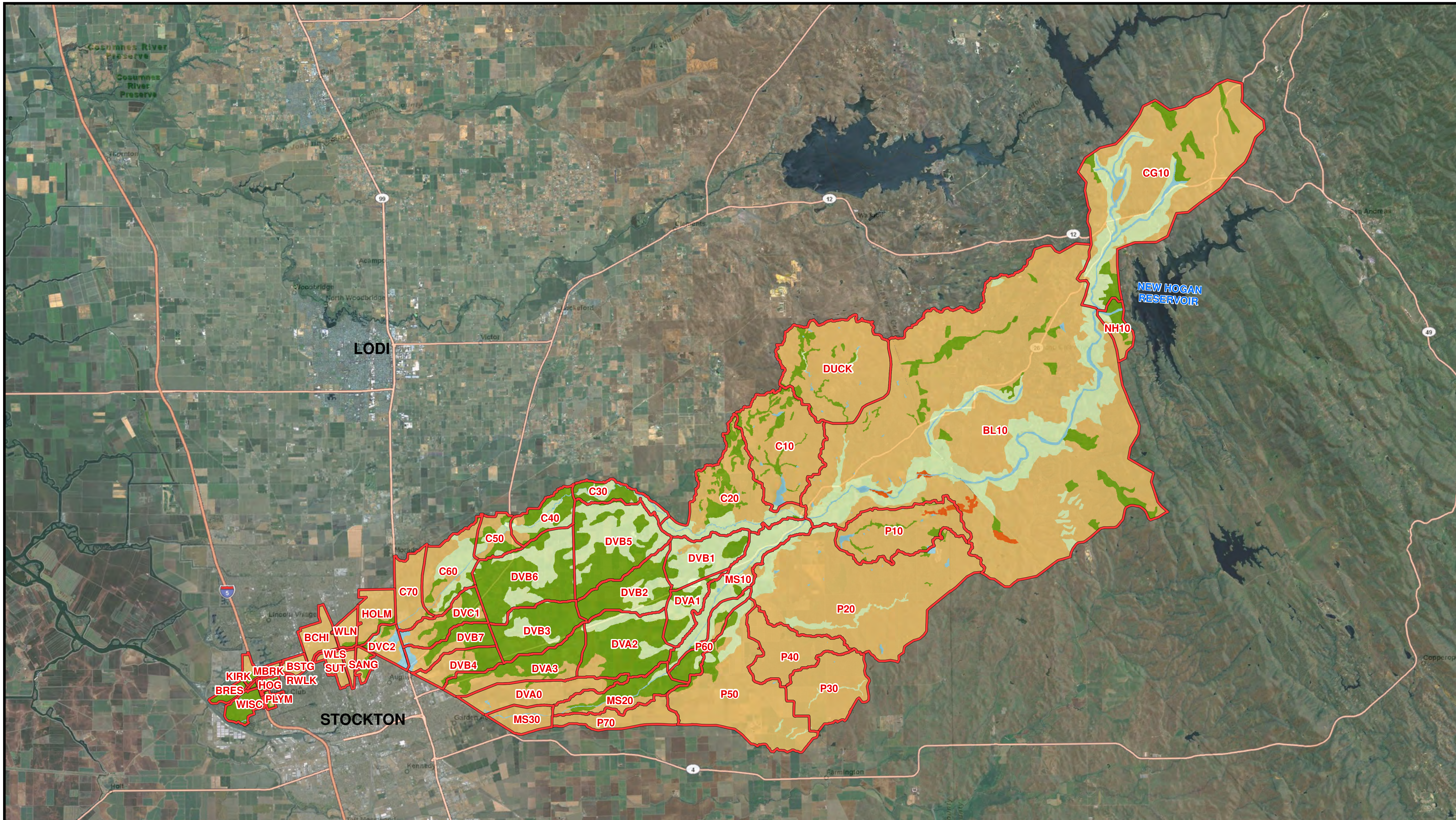
Table 5- 4. NRCS hydrologic soil groups.

Hydrologic Soil Group	Loss Rate Range [in/hr]	PBI's Assumed Loss Rate[a] [in/hr]
A	> 0.30	0.35
B	0.15 - 0.30	0.2
C	0.05 - 0.15	0.1
D	0.00 - 0.05	0.025

[a]This loss rate value was assigned to each soil group for initial calculations of composite loss rates. The calculated composite loss rates were then adjusted during the calibration process.

GIS soils data was obtained from the NRCS[13] and used to determine the proportional coverage of soil groups within Calaveras River subbasins (Figure 5- 7). NRCS GIS soils data was not available for Calaveras County. Soils data for this part of the study area was obtained from the Calaveras County Soil-Vegetation Survey[14]. A weighted average of loss rates was then calculated for each subbasin and adjusted during the calibration process (See Section 5.4). After the calibration adjustment, subbasin loss rates range from 0.021 inches per hour to 0.158 inches per hour as shown in Attachment 5-C.

EM 1110-2-1417[18] recommends that initial losses are set between 0.5-1.5 inches for agricultural areas. Initial losses were set to 0.5 inches for all agricultural/rural subbasins in the foothills and to 1.5 inches for agricultural/rural subbasins in the valley. For urban subbasins, initial losses were set to 0.2 inches also based on guidelines listed in *EM 1110-2-1417*.



Subbasin Boundary
Group A
Group B
Group C
Group D
Water/Other

0 1.5 3 Miles

1 inch = 3 miles

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CALAVERAS RIVER WATERSHED SOILS MAP

FIGURE

5-7

5.3.8. Impervious Percentages

Impervious percentages were assigned based on the extent of urbanization within each subbasin. Aerial photos including those contained within 2010 LiDAR datasets[4] were used to assess existing urbanization in the Calaveras River watershed. Subbasins were classified into several categories with assigned impervious percentages as shown in Table 5- 5. The impervious percentages corresponding to each land use type were selected with the guidance of the San Joaquin County *Hydrology Manual*[10].

Table 5- 5. Land use types and their corresponding impervious percentages.

Land Use Type	Impervious Percentage
Agricultural /Open Space	2%
Agricultural with Rural Residential Development	5%
Fully Developed Residential	60%

5.4. MODEL CALIBRATION

HDR's Calibration of the 1998 SJAFCA HEC-1 model included calculating the flow per square mile for the 1/100 AEP event at the Duck Creek near Farmington gage and comparing it to the modeled flow per square mile coming from the foothill subbasins that feed into Potter Creek. Adjustments were made to basin 'n' values such that the 1/100 AEP rainfall event would produce the 1/100 AEP streamflow event.

The PBI Model was calibrated to an observed rainfall-runoff event using gaged data retrieved from the California Data Exchange Center (CDEC)[24]. The three gages shown in Figure 5- 2 were used for the calibration: the Perry Ranch (PRY) gage was used for its rainfall data, the New Hogan Lake (NHG) gage was used for its reservoir outflow data, and the Mormon Slough at Bellota (MRS) gage was used for its flow records.

The storm selected to calibrate the PBI Model was one of the largest events recorded by the MRS gage. The rainfall event took place between March 25, 2006 and April 16, 2006 (23-day duration) and totaled 7.9 inches.

The MRS gage location corresponds to Model Element BL15. During the calibration process, constant loss rates were adjusted to match the PBI Model's hydrograph at Model Element BL15 with observed streamflow records from the MRS gage. Constant loss rates were initially calculated based on the makeup of soils in each subbasin (see Section 5.3.7). The loss rates were then adjusted by a factor of 0.85 during the calibration process.

Figure 5- 8 shows the observed New Hogan outflow during the calibration storm event. The results of the calibration are shown in Figure 5- 9.

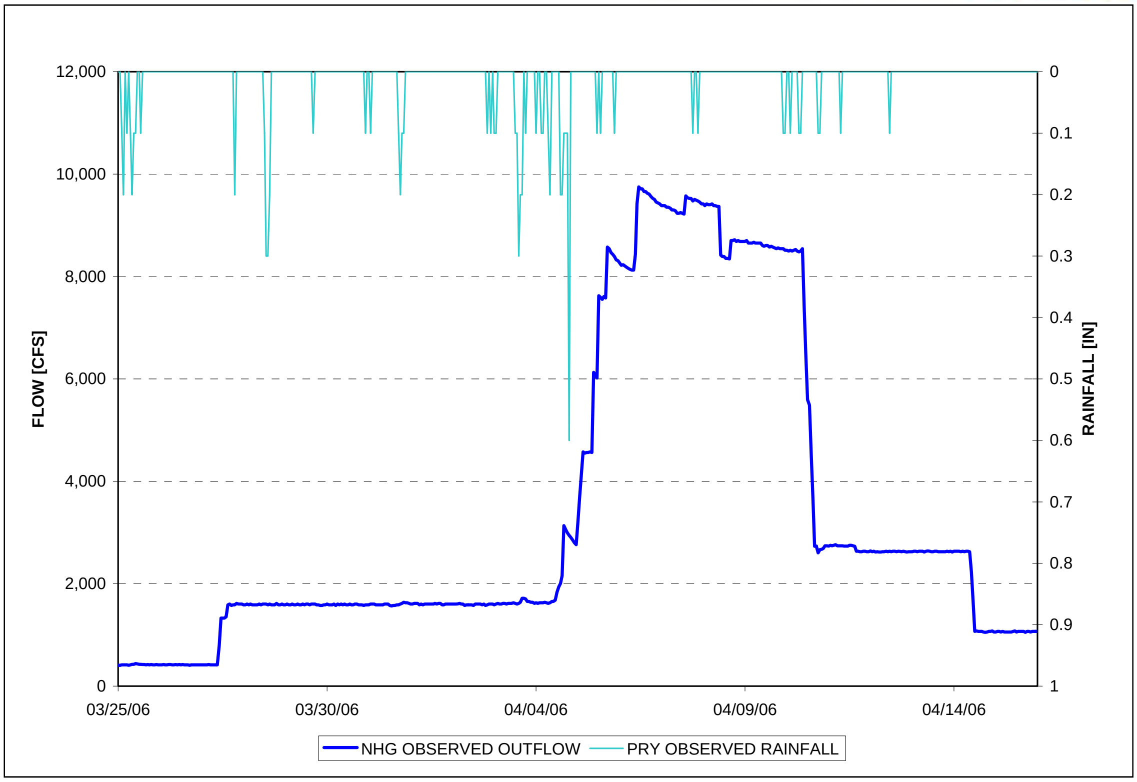


Figure 5- 8. Observed New Hogan Outflow During the Calibration Storm Event.

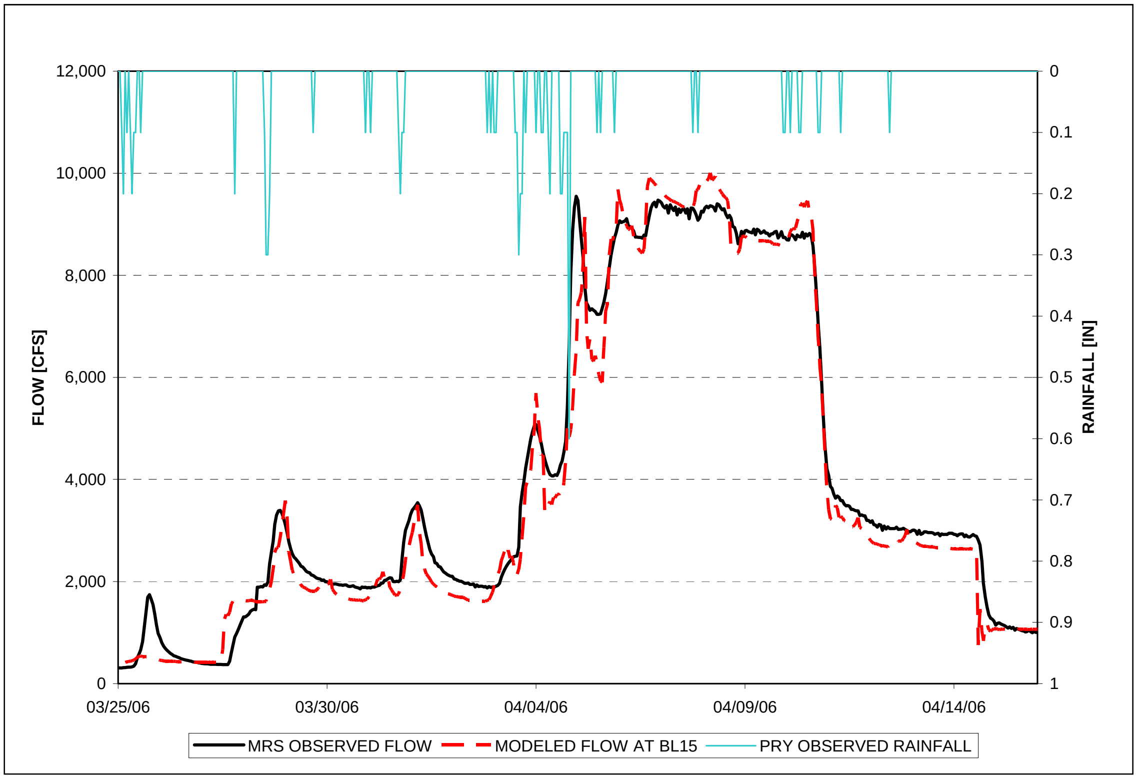


Figure 5- 9. Observed versus Modeled Flow at Bellota for the Calibration Storm Event.

At the onset of the storm, the initial runoff response is not picked up by the HEC-HMS model. This is due to the initial losses (see Section 5.3.7) assigned to the subbasins upstream of the MRS gage. These subbasins are largely undeveloped with little impervious area and therefore the soils capture the initial rainfall. Although this runoff response could be captured by decreasing the initial losses, initial loss was held at their assigned values based on the ranges suggested in *EM-1417*[18]. Furthermore, the emphasis of the hydrologic analysis is on peak event estimation which is relatively insensitive to initial loss assumptions.

5.5. DEVELOPMENT CONDITIONS

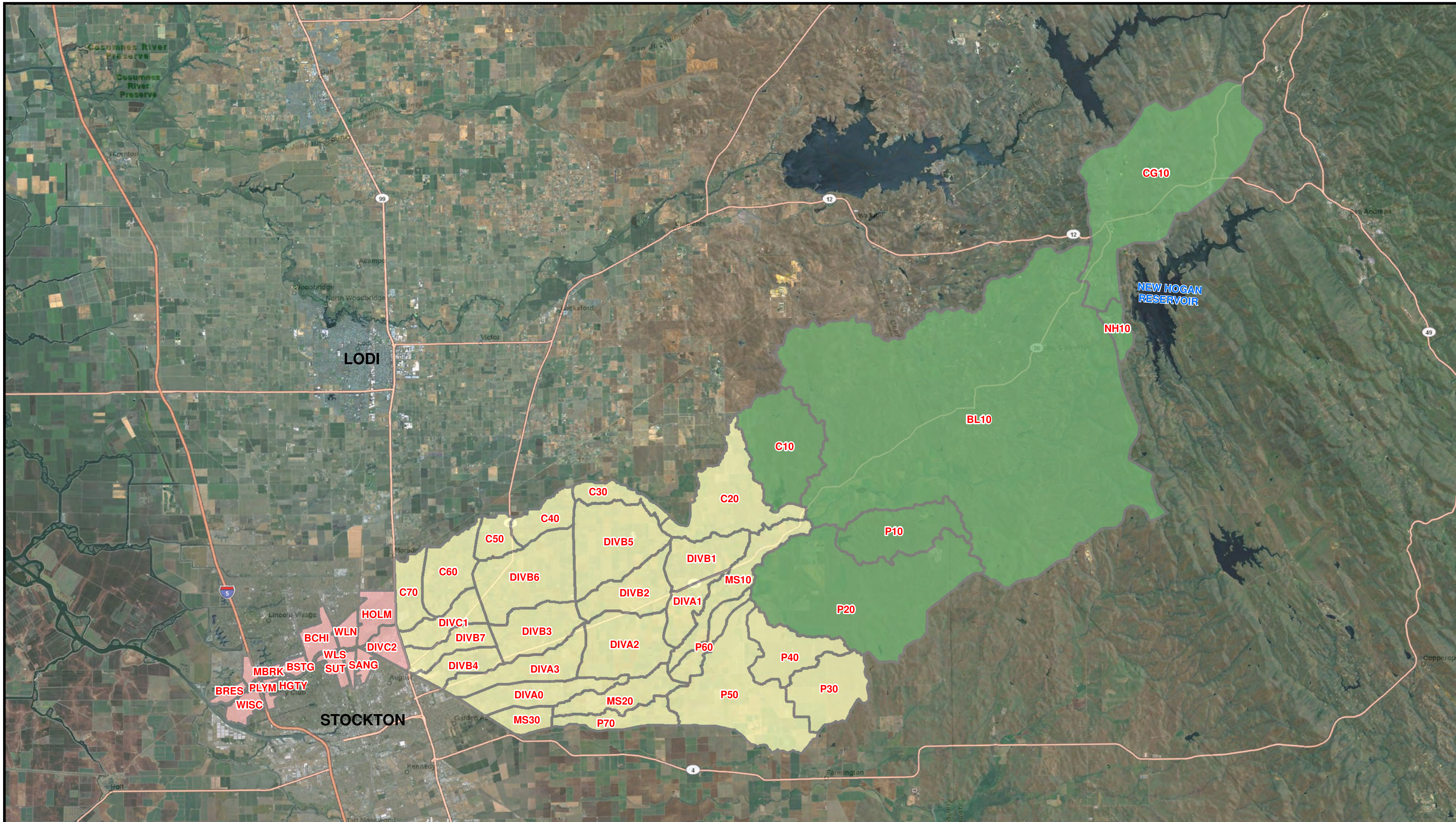
5.5.1. Existing Conditions

An 'Existing Conditions' model run was performed to evaluate peak flows given current (2010) land use and hydrologic conditions within the Calaveras watershed. Subbasin S-graphs, 'n' values, and impervious percentages were set according to current land cover conditions using field knowledge supplemented by aerial photos.

In general, the upstream watershed consists of natural or agricultural land whereas the lower portions of Calaveras watershed (below the confluence with the Diverting Canal) are developed areas in and around the city of Stockton. Figure 5- 10 displays the development conditions and S-graphs assigned to each subbasin. A summary table of the subbasin characteristics used for 'Existing Conditions' model runs is provided in Attachment 5-B.

5.1.1. Future-Without-Project Conditions

A 'Future-Without-Project Conditions' model run was considered to evaluate peak flows for future (2070) land use and hydrologic conditions within the Calaveras River watershed. However, the City of Stockton 2035 General Plan[12] and the San Joaquin County General Plan[17] show that land use remains unchanged from the 'Existing Conditions' model. Because of this, the 'Future-Without-Project' model simulations will be identical to 'Existing Conditions' simulations.



Foothill S-Graph
Valley Undeveloped S-Graph
Valley Developed S-Graph

N

0 1.5 3 Miles
1 inch = 3 miles

SEPTEMBER 21, 2010

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EXISTING DEVELOPMENT CONDITIONS FOR CALAVERAS RIVER WATERSHED

FIGURE 5-10

5.2. STORM CENTERINGS

Four storm centerings were analyzed for the Calaveras River watershed:

- The *Above New Hogan* centering was analyzed to stress both New Hogan Dam and the watershed below.

- The *Bellota* centering was analyzed to stress the unregulated portion of the watershed directly below the dam.

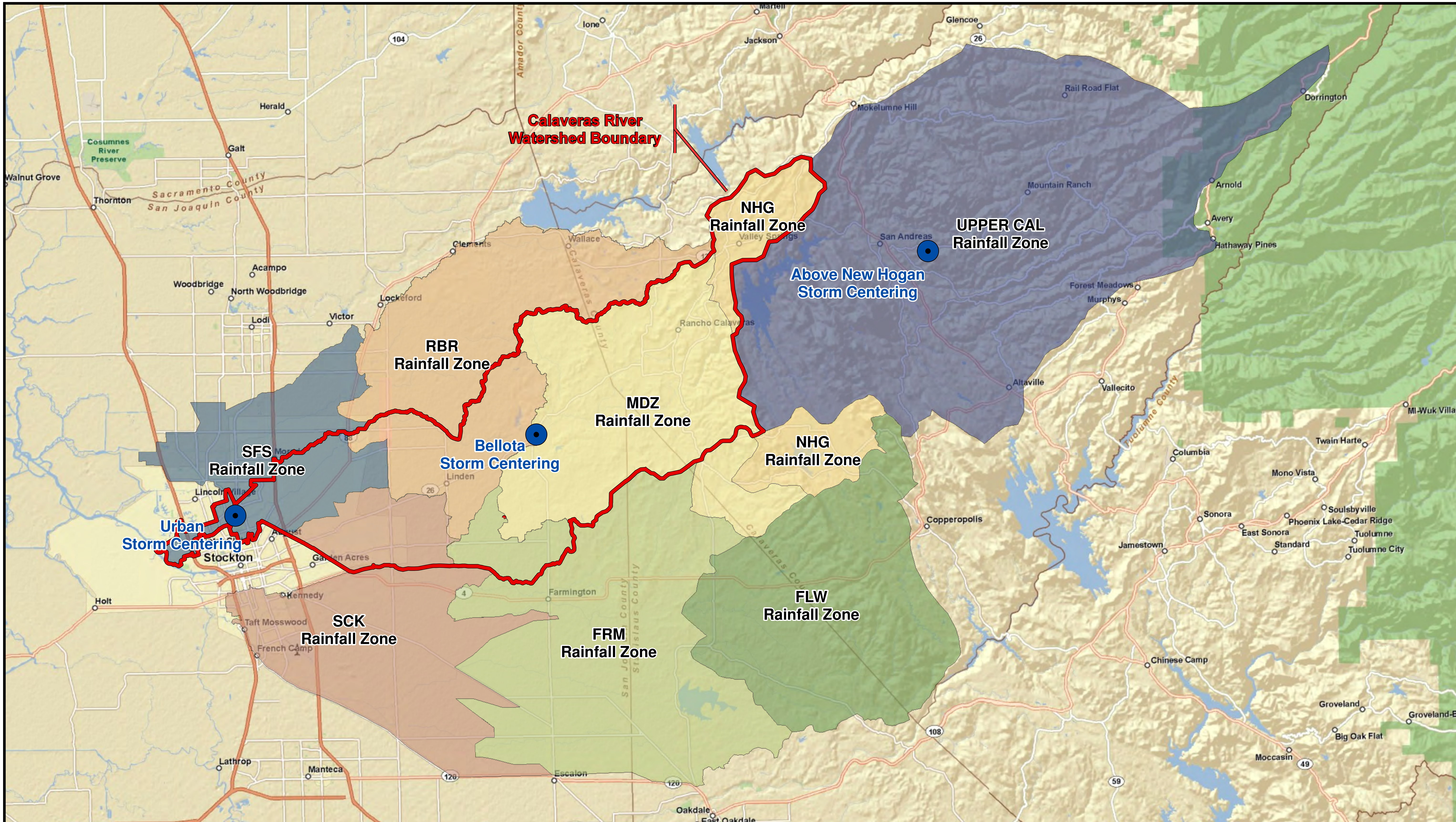
- The *"Average"* centering took the average of the *Above New Hogan* and *Bellota* area reduction factors to come up with rainfall depths.

  Reservoir flows from the Ford analysis were reported for an "Average" storm centering and are most applicable for this scenario. The "Average" storm centering scenario is considered the LSJRFS design storm scenario for the Calaveras River production runs and therefore are the only flows reported in Table 5- 7.

- The *Urban* centering was analyzed for interior drainage purposes and is directly centered over the urban areas in the lower watershed.

Eight design storms with the 1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, and 1/500 AEP events will be produced for each centering. This selection of design storms provides a wide range of scenarios that can be used for planning purposes.

Calculated area reduction factors and resulting area-reduced rainfall depths for each rainfall zone are provided in Attachment 5-D for all frequency-duration-storm centering combinations.



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LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

FIGURE 5-11

CALAVERAS RIVER WATERSHED STORM CENTERINGS

5.3. MODEL SIMULATIONS

Calaveras River production runs include 32 scenarios with unique combinations of development conditions, storm frequencies, and storm centerings.

Table 5- 6. Calaveras River production run scenarios.

Development Conditions	Storm Centerings	AEP Events
Existing Conditions	Urban	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	Bellota	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	"Average"	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	Above New Hogan	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500

5.3.1. Summary of Results

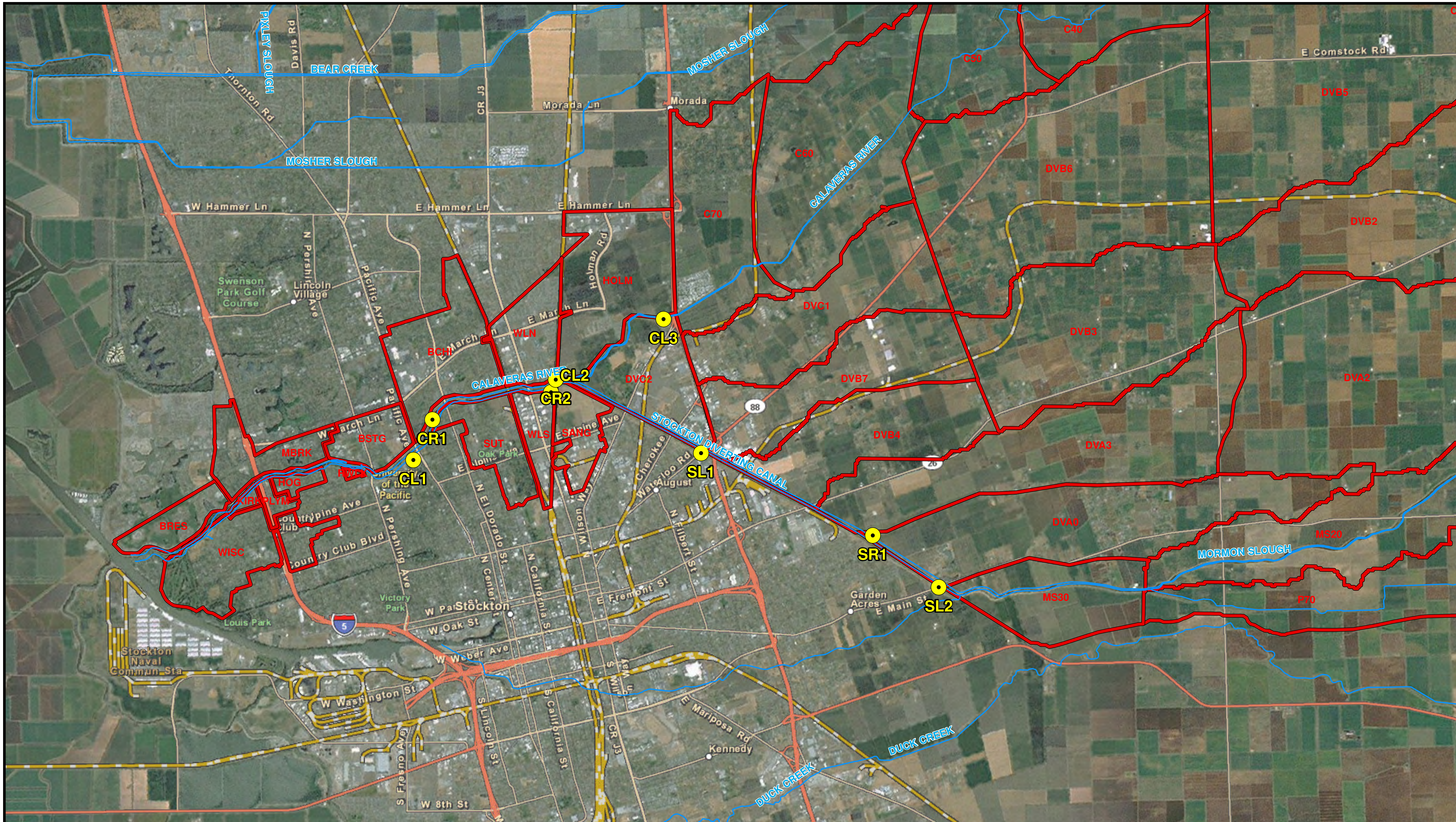
Peak flow results were extracted from HEC-HMS at each LSJRFS index point. Locations of LSJRFS index points within the Calaveras River watershed are shown in Figure 5- 12. Table 5- 7 summarizes peak flows from the "Average" storm centering production runs.

Reservoir flows from the Ford analysis were reported for an "Average" storm centering and are most applicable for this scenario. The "Average" storm centering scenario is considered the LSJRFS design storm scenario for the Calaveras River production runs and therefore are the only flows reported in Table 5- 7. Flows produced from this storm centering are to be used moving forward with the LSJRFS.

5.3.2. Uncertainty Parameters

For the purposes of the LSJRFS, uncertainty parameters for each flow-frequency dataset can be estimated within HEC-FDA during the project's economic analysis. HEC-FDA defines uncertainty in terms of confidence intervals or standard deviations given inputs of flow-frequency data (provided in Table 5- 7) and an equivalent record length.

The equivalent record length is an estimate of the overall "worth" or "quality" of the flow-frequency function, expressed as the number of years-of-record[19]. For probability functions derived at ungaged locations using model or other data, the equivalent record length is based on a judgment of the quality of that model or data. EM 1110-2-1619[20] provides guidelines for assigning equivalent record lengths and estimates that a rainfall-runoff model calibrated to an observed event at a short-interval runoff gage has an equivalent record length of 20-30 years.



	Subshed Boundary
	LSJRFS Index Point

N

0 0.25 0.5 1 Miles

1 inch = 1 mile

JUNE 20, 2012

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LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

CALAVERAS RIVER WATERSHED INDEX POINTS

FIGURE 5-12

Table 5-7. Peak Flow Results for Calaveras River - Existing and Future Conditions [cfs] [1]

LSJRFS Index Point ID	Description	Average Storm Centering							
		1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP
--	Calaveras R. at Duncan Rd.	110	230	300	410	500	580	670	780
CL3	Calaveras River near Hwy 99	110	230	300	440	530	620	720	810
--	Mormon Slough at Bellota[2]	3,520	9,520	9,390	10,320	12,500	12,500	12,500	16,000
--	Mormon Slough at Potter A Confl.	4,150	10,150	10,630	12,130	14,200	14,940	15,280	19,460
SL2	Mormon Slough at Diverting Canal	4,150	10,150	10,620	12,140	14,210	14,960	15,320	19,510
SR1	Diverting Canal u/s of Hwy 26	4,150	10,150	10,630	12,150	14,220	14,970	15,340	19,530
SL1	Diverting Canal near Hwy 99	4,150	10,150	10,670	12,230	14,320	15,070	15,440	19,620
CR2 & CL2	Calaveras River at Diverting Canal Confl.	3,810	9,620	10,050	12,530	13,670	15,650	16,110	20,230
CR1	Calaveras R. d/s of El Dorado St.	3,700	9,660	9,780	12,520	13,320	15,610	16,100	20,190
CL1	Calaveras R. d/s of Pacific Ave.	3,700	9,660	9,780	12,520	13,320	15,610	16,100	20,190
D4 & D5	Calaveras R. u/s of San Joaquin R. Confl.	3,560	9,520	9,760	12,390	12,780	15,600	16,100	20,160

[1]There were no changes from Existing to Future conditions, therefore only one results table is shown.

[2]Input hydrographs at Bellota provided to PBI by USACE on 2/1/12.

6.0 FRENCH CAMP SLOUGH HEC-HMS MODELING

6.1. GENERAL

6.1.1. Location

The French Camp Slough watershed is located near the city of Stockton in San Joaquin County, California (Figure 6- 1). The watershed runs east from the city of Stockton into the Sierra Nevada foothills in Calaveras County. It achieves maximum elevations of 2,100 feet and includes a total area of 430 square miles. It then descends through moderate slopes to the lower portion of the watershed which lies at sea-level. None of the watershed experiences snowfall; all floods are rainfall-induced.

The HEC-HMS model described in this memorandum includes the Duck Creek, Lone Tree Creek, Temple Creek, Rock Creek, Webb Creek, Littlejohns Creek, and the French Camp Slough systems and discharges to the San Joaquin River to the west of Interstate-5.

6.1.2. Topography

The HEC-HMS model utilized for this study is titled the PBI French Camp Slough Model (PBI Model) which is georeferenced to the NAD 1983 State Plane California Coordinate System Zone III (U.S. Survey Feet). Vertical elevations are reported in the NAVD 1988 datum. Topography used for model development included United States Geological Survey (USGS) 30-meter Digital Elevation Models (DEMs)[3]. Where available, Department of Water Resources (DWR) LiDAR data[4] was also used to confirm subbasin boundaries.

6.2. MODEL DEVELOPMENT

The PBI model was developed using HEC-HMS version 3.4[5] and HEC-GeoHMS version 4.2[6]. A summary of the tasks performed are listed below:

1. A previous HEC-HMS model used in the *Conditional Letter of Map Revision (CLOMR) for the Tidewater Crossing Flood Control Project*[25] (Tidewater Model) provided the basis for the PBI Model (See Section 6.2.1).

2. Additional subbasins were added to the Tidewater model. Eleven (11) subbasins on Duck Creek and North Littlejohns Creek were imported from a previous HEC-1 model constructed as part of the *Mariposa Lakes Off-Site Regional Hydrologic Investigation*[26] (Mariposa Lakes Model) (See Section 6.2.2). Eighteen (18) additional subbasins extended the PBI Model to French Camp Slough's outlet on the San Joaquin River.

3. Pump stations were coded into the PBI Model based on design pumping rates

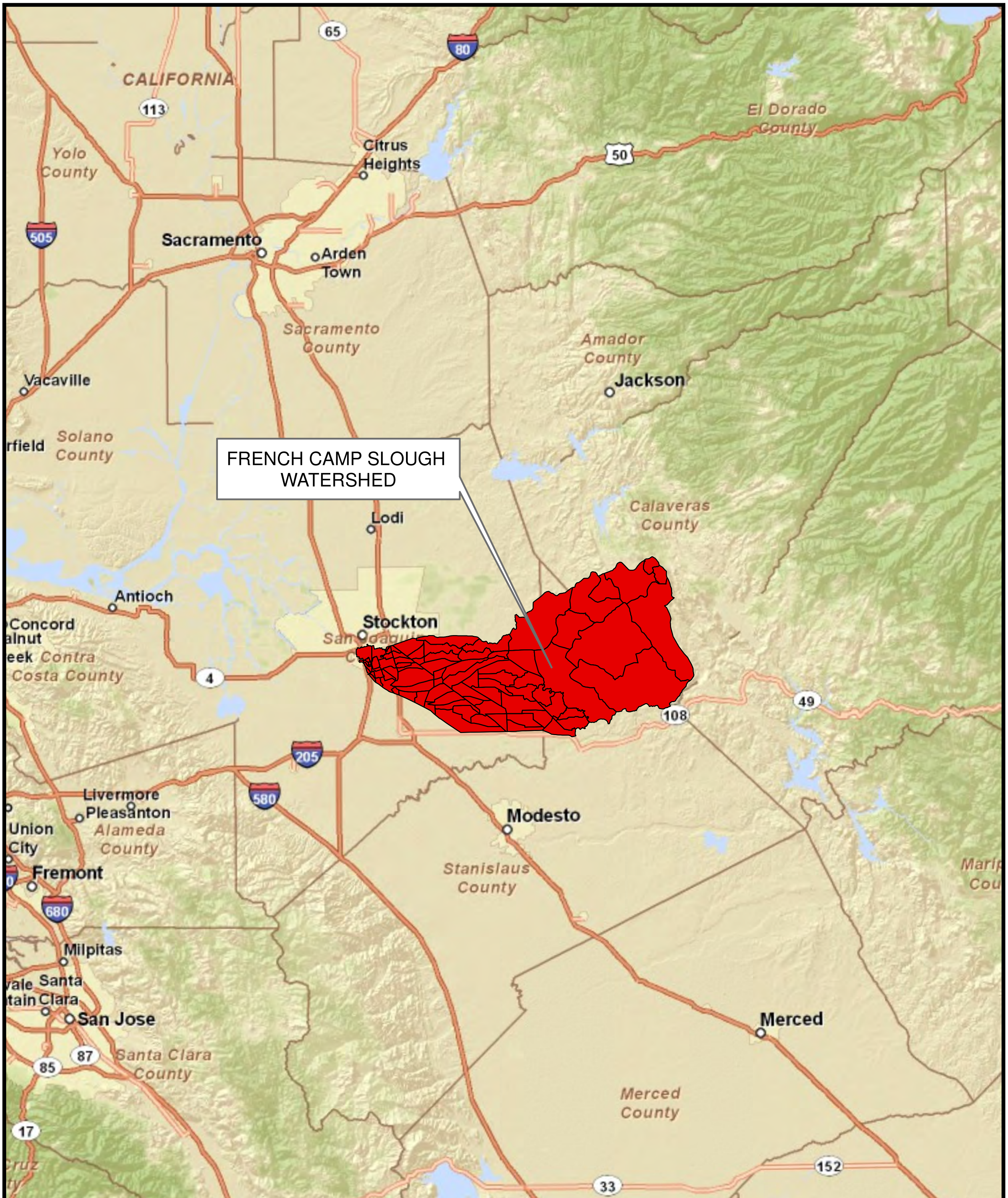

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FIGURE 6-1

VICINITY MAP OF THE FRENCH CAMP SLOUGH STUDY AREA

provided by City of Stockton records[7] (See Section 6.3.2).

4. Diversions and channel routing parameters were coded for the added subbasins of the PBI Model (See Sections 6.3.4 and 6.3.6, respectively).

5. S-graphs and lag times were coded into the PBI Model (See Section 6.3.5).

6. Loss rates and impervious percentages were coded into the PBI Model (See Section 6.3.7 and Section 6.3.8).

7. The PBI Model was calibrated using historical rainfall and runoff data (See Section 6.4).

8. The PBI Model was set up to simulate both 'Existing' (see Section 6.5.1) and 'Future-Without-Project' (see Section 6.5.2) scenario runs.

6.2.1. Tidewater HEC-HMS Model

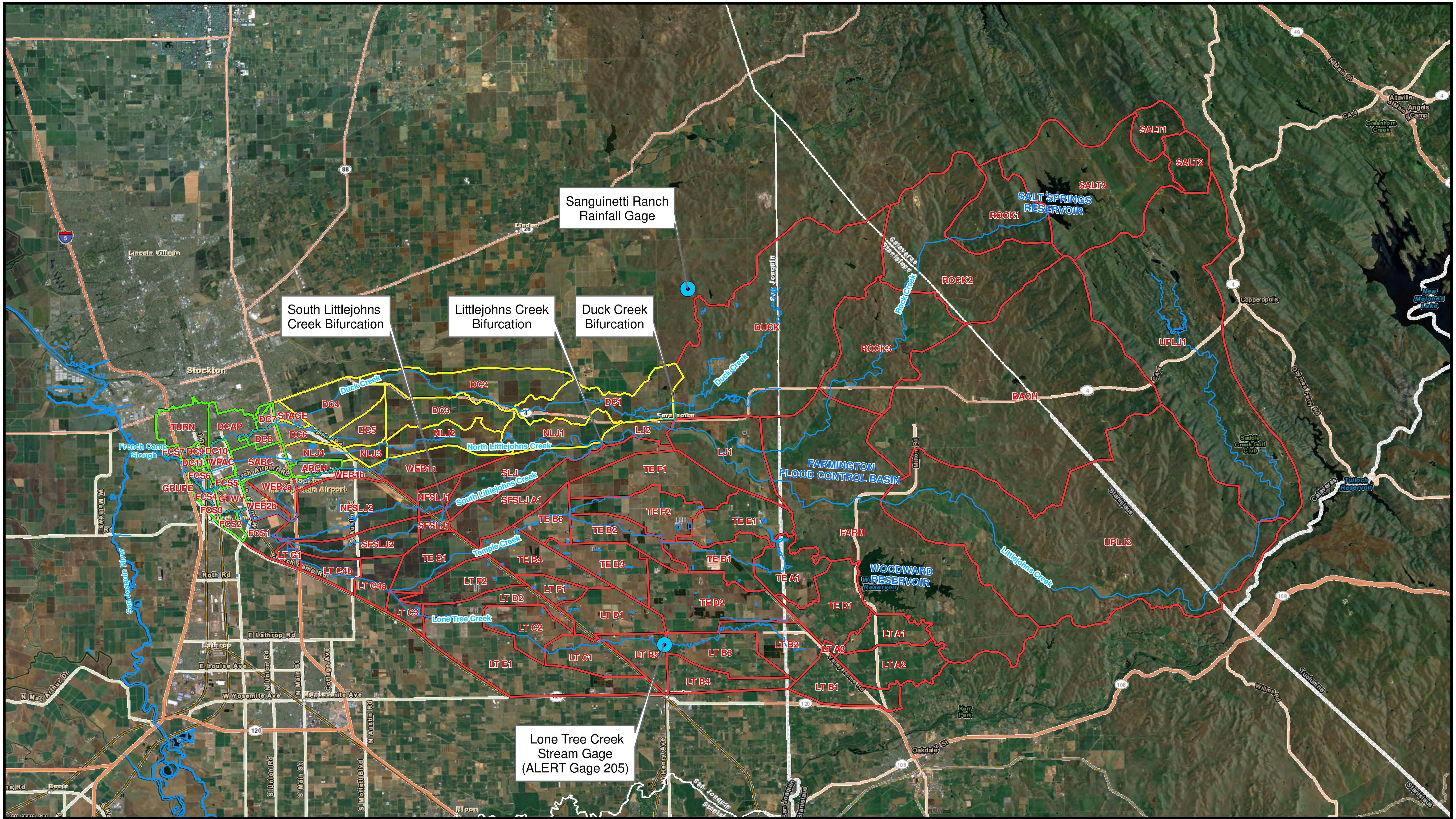
The PBI Model is an update and expansion of the HEC-HMS model developed in 2007 for the *Conditional Letter of Map Revision (CLOMR) for the Tidewater Crossing Flood Control Project*[25].

The 2007 Tidewater Model includes 55 subbasins which are coded with SCS Curve Numbers, San Joaquin County S-Graphs, and calculated lag times. Reach routing, diversions, and reservoir routing were also coded into the Tidewater Model based on field visits conducted by Domenichelli & Associates.

Calibration of the Tidewater Model used a January 1, 2006 storm which was estimated to be a 1/10 AEP event. This enabled observed high water marks on Lone Tree Creek to be calibrated to recorded rainfall data.

6.2.2. Mariposa Lakes HEC-1 Model

A HEC-1 model was previously developed in 2006 as part of the *Mariposa Lakes Off-Site Regional Hydrologic Investigation*[26]. PBI extracted 11 subbasins from the Mariposa Lakes HEC-1 Model and imported them into the Tidewater HEC-HMS model. The imported Mariposa Lakes model elements are located on Duck Creek and North Littlejohns Creek systems.



Red	Subbasins from the Tidewater Model
Yellow	Subbasins from the Mariposa Lakes Model
Green	Subbasins Added by PBI

N

0 1.5 3 Miles

1 inch = 3 miles

OCTOBER 19, 2010

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FRENCH CAMP SLOUGH HEC-HMS SUBBASINS

FIGURE 6-2

6.3. MODEL FEATURES

The 2007 Tidewater HEC-HMS model was updated and expanded to form the PBI French Camp Slough Model. The PBI Model components are described in the following sections.

6.3.1. Subbasins

The PBI Model contains a total of 85 subbasins with drainage areas ranging from 0.04 square miles to 51.62 square miles and a total watershed area of approximately 430 square miles. Figure 6- 2 displays the subbasin boundaries used for the PBI Model.

As previously discussed, 55 of the PBI Model's subbasins come from the 2007 Tidewater Model. These subbasin boundaries include 385 square miles of tributary area and cover much of the Duck Creek, Lone Tree Creek, Temple Creek, Webb Creek, Rock Creek, Littlejohns Creek, and the French Camp Slough systems.

The calculated areas of several subbasins on Lone Tree Creek and Temple Creek were adjusted to account for parts of the subbasin that were considered to be isolated and not contributing to runoff. These adjustments were based on field investigations conducted for the Tidewater Model which determined that ponding in fields would occur and this ponded area would not contribute to the subbasins' modeled runoff. A summary of the adjusted subbasin areas is provided in Attachment 6-A.

Eleven (11) subbasins totaling 31 square miles were extracted from the Mariposa Lakes HEC-1 Model. These subbasins cover portions of the Duck Creek and North Littlejohns Creek systems.

Eighteen (18) subbasins totaling 12 square miles were added by PBI which extend the model boundaries to near French Camp Slough's outlet on the San Joaquin River. Subbasins to the west drain to the San Joaquin River. Many of these subbasin boundaries were based on the existing storm drain system and the City of Stockton's *Conceptual Storm Drain Master Plan*[11]. Many of these subbasins discharge to the main channels through various stormwater pump stations described in Section 3.3.2. Areas that are not drained through the storm sewer system are gravity-driven. USGS 30-meter DEM datasets[3] were used to identify local topography for delineating gravity-driven subbasins. Where available, DWR LiDAR[4] data was used to confirm subbasin boundaries.

The GIS horizontal coordinates for each subbasin were used to georeference model elements within the PBI HEC-HMS Model. The subbasin GIS shapefile was inserted into the PBI Model as a background map.

6.3.2. Pump Stations

Stormwater pump stations were included in the PBI Model to represent storm drain conveyance from developed subbasins to the main channels. There are nine (9) pump stations included in the PBI model with capacities assigned based on City of Stockton records[7].

Table 6-1 provides a summary of the City of Stockton pump stations included in the PBI Model.

Table 6- 1. Summary of French Camp Slough pump stations.

Pump Station Name	Contributing Subbasin(s)	Subbasin Area [sq. mi.]	Pump Station Status	Pump Station Capacity [cfs]	Pump Station Notes
Stockton Airport Business Center	SABC	1.80	Existing	66.8	4 @ 15.4 cfs 1 @ 5.2 cfs
Duck Creek & Airport Way	DCAP	1.79	Existing	114.8	1 @ 5.6 cfs 1 @ 49.0 cfs 1 @ 60.2
Arch Road	ARCH	0.74	Existing	10.8	2 @ 5.4 cfs
Clayton & Harvey	CLAY	0.10	Existing	9.8	2 @ 4.9 cfs
Grupe Business Park	GRUPE	0.20	Existing	120.7	1 @ 11.1 cfs 2 @ 54.8 cfs
Duck Creek & Stagecoach	STAGE	0.50	Existing	155.4	1 @ 5.6 cfs 2 @ 74.9 cfs
Walker Slough & Turnpike	TURN	2.33	Existing	116.9	1 @ 8.0 cfs 2 @ 37.4 cfs 1 @ 34.1 cfs
Western Pacific Industrial Park	WPIP	0.93	Existing	60.7	1 @ 2.7 cfs 2 @ 29.0 cfs
Airport Gateway	GTWY	0.77	Existing	22.3	1 @ 1.9 cfs 2 @ 10.2 cfs
PS-DC10	DC10	0.04	Future	9.5	Based on 0.37 cfs/acre
PS-DC11	DC11	0.23	Future	54.5	Based on 0.37 cfs/acre
PS-DC4	DC4	3.80	Future	899.8	Based on 0.37 cfs/acre
PS-DC5	DC5	1.68	Future	397.8	Based on 0.37 cfs/acre
PS-DC6	DC6	0.92	Future	217.9	Based on 0.37 cfs/acre
PS-DC7	DC7	0.32	Future	75.8	Based on 0.37 cfs/acre
PS-DC8	DC8	0.62	Future	146.8	Based on 0.37 cfs/acre
PS-DC9	DC9	0.17	Future	40.3	Based on 0.37 cfs/acre
PS-FCS1	FCS1	1.70	Future	402.6	Based on 0.37 cfs/acre
PS-FCS2	FCS2	0.46	Future	108.9	Based on 0.37 cfs/acre
PS-FCS3	FCS3	0.26	Future	61.6	Based on 0.37 cfs/acre
PS-FCS4	FCS4	0.20	Future	47.4	Based on 0.37 cfs/acre

con't Table 6- 1…

Pump Station Name	Contributing Subbasin(s)	Subbasin Area [sq. mi.]	Pump Station Status	Pump Station Capacity [cfs]	Pump Station Notes
PS-FCS5	FCS5	0.30	Future	71.0	Based on 0.37 cfs/acre
PS-FCS6	FCS6	0.38	Future	90.0	Based on 0.37 cfs/acre
PS-FCS7	FCS7	0.12	Future	28.4	Based on 0.37 cfs/acre
PS-LT C4b	LT C4b	1.19	Future	281.8	Based on 0.37 cfs/acre
PS-LT G1	LT G1	0.45	Future	106.6	Based on 0.37 cfs/acre
PS-NFSLJ2	NFSLJ2	6.78	Future	1605.5	Based on 0.37 cfs/acre
PS-NLJ3	NLJ3	0.75	Future	177.6	Based on 0.37 cfs/acre
PS-NLJ4	NLJ4	1.19	Future	281.8	Based on 0.37 cfs/acre
PS-SFSLJ2	SFSLJ2	3.30	Future	781.4	Based on 0.37 cfs/acre
PS-Web1b	Web1b	1.11	Future	262.8	Based on 0.37 cfs/acre
PS-Web2a	Web2a	1.42	Future	336.3	Based on 0.37 cfs/acre

Twenty-three pump stations were then added into the 'Future-Without-Project Conditions' model to represent subbasins that are expected to become developed according to the City of Stockton 2035 General Plan[12]. Pump capacities were assigned at a rate of 0.37 cfs per acre of tributary area. This rate is based on the average flow rates of existing pump stations within the City of Stockton's systems and correlates to approximately 10-year peak flows.

6.3.3. Reservoirs

There are three main reservoirs in the PBI Model study area: Salt Springs Reservoir, Woodward Reservoir, and Farmington Flood Control Basin.

Salt Springs Reservoir

The Salt Springs Reservoir is a small reservoir that impounds flow on Rock Creek and is primarily used for recreation. A field study conducted for the Mariposa Lakes model confirmed that water simply spills over this small concrete dam structure when the reservoir is full[26]. Inflow roughly equals outflow and the hydraulic effects of this reservoir become negligible.

Woodward Reservoir

Woodward Reservoir is operated by South San Joaquin Irrigation District (SSJID) and releases water directly into a SSJID irrigation canal. The Tidewater model assumed that during the flood season (November-March) Woodward Reservoir's tributary area drains towards the Farmington Flood Control Basin and through Farmington Dam[25]. A telephone conversation with a SSJID engineer confirmed that Woodward Reservoir is an off-stream reservoir and any major releases are limited to the irrigation season (April-October). There is no spillway associated with this reservoir and any overtopping during the flood season would follow the natural topography of the land traveling through Simmons Slough and over towards Farmington Flood Control Basin.

Farmington Flood Control Basin

Farmington Reservoir is a large flood control basin located about 20 miles east of Stockton and impounds flow from both Rock Creek and Littlejohns Creek. The dam itself is approximately 7,800 feet long and 58 feet high with two outlets controlled by slide gates.

David Ford Consulting Engineers (DFCE) completed a separate reservoir operations analysis for Farmington Reservoir as part of the LSJRFS[23]. This analysis was later amended by USACE as documented in their *Draft Memorandum for Record: Lower San Joaquin River Feasibility Study, Bellota and Farmington Regulated Flow Hydrographs* (07 FEB 2012)[22]. One of the final deliverables from this study was regulated hydrographs at the Farmington control point for each of the 8 LSJRFS AEP storm events. These hydrographs include all flows coming out of Farmington Dam along with all local flows between the Town of Farmington and Farmington Dam. These regulated flow hydrographs were coded into the PBI HEC-HMS model as time-series discharge gages and supersede all HEC-HMS inflow that comes from above the Town of Farmington.

The Ford report and the USACE amendment should be referenced for any details regarding the reservoir operations study.

The table on the following page was taken from the USACE amendment and shows the flow-frequency relationship for Littlejohn Creek at the Farmington control point.

Table 6- 2. Flow-frequency at Farmington Reservoir (from Ford Report[23])

Regulated Peak Flow values and associated volumes: Littlejohn Creek at Farmington					
Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes[1] (as average flow for given duration) 1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)
0.5	1,400	1,206	1,041	797	550
0.2	2,170	1,870	1,796	1,614	1,138
0.1	2,368	2,018	1,921	1,756	1,426
0.04	2,615	2,089	2,002	1,839	1,736
0.02	3,744	3,486	2,070	1,900	1,843
0.01	9,900	8,600	7,400	5,400	3,800
0.005	12,900	12,000	10,000	7,400	4,400
0.002	16,600	15,200	12,000	8,600	5,200

1) Revised to reflect graphical fit of observed data from Oct1949 to Dec2011 for the 0.5 to the 0.02 AEP. The 0.01 to 0.002 AEP events are from the revised flow transform and regulated flow-freq curve. The volumes were computed from the regulated peak to volume transforms in the Ford report.

Miscellaneous Reservoir Elements

Along with the three reservoirs discussed, several additional reservoir elements were included in the PBI Model to represent flow restrictions as channels encounter road and railway crossings. For development of the Tidewater Model, measurements were taken of culvert and bridge geometry. Rating curves estimating the hydraulic performance of many crossings in the mid- and upper watershed were determined by entering measured geometries into HEC-RAS and simulating a range of flows through the structures. Hydraulic calculations associated with assigning reservoir storage/discharge relationships were performed for the 2007 Tidewater Model and are included in Attachment 6-B. The computed flow relationships were incorporated into reservoir elements to represent flow impedance at the selected road and railway crossings.

Some reservoirs include elevation-storage functions in which the elevations were reported in the NGVD29 coordinate system. To stay consistent with current conventions, the elevations were converted to the NAVD88 coordinate system using CORPSCON v6.0.1[27] software. A summary of the conversion is provided in Attachment 6-C.

6.3.4. Diversions

Diversions in HEC-HMS are coded to simulate either manmade diversions or topographic flow splits. Twenty-five (25) diversions are included in the PBI Model were imported from the Tidewater and Mariposa Lakes hydrologic models.

There are three (3) diversions used to represent channel bifurcations in the PBI Model. Channel bifurcations occur on Duck Creek, Littlejohns Creek, and South Littlejohns Creek as seen in Figure 6- 2. Coding for the Duck Creek and South Littlejohns Creek diversions was taken from the Tidewater HEC-HMS model whereas the Littlejohns Creek diversion coding was taken from the Mariposa Lakes HEC-1 model. The Duck Creek bifurcation has a structure to control the flow diverted to Littlejohns Creek whereas diversion flows for the Littlejohns Creek and South Littlejohns Creek bifurcations were proportionally based on channel geometries.

The remaining twenty-two (22) diversions included in the PBI Model are used to represent topographic flow splits at road and railway crossings and were imported from the Tidewater HEC-HMS model. As mentioned in Section 6.3.3, road and railway crossings were modeled using reservoir elements. For the cases where floodwaters in the overbank areas are unable to return to the main channel due to berms and other impedances, a diversion element was utilized to take this excess water and route it through an additional reservoir element. This reservoir element then routes the excess flow appropriately through small pipes or overland surfaces as it eventually returns back to the main channel.

6.3.5. S-graphs and Lag Times

The PBI Model assigns a Foothill, Valley Undeveloped, or Valley Developed S-graph to each subbasin based on its location. S-graph data points were obtained from the San Joaquin County Hydrology Manual[10]. The S-graphs were developed based on rainfall-runoff data from Southern California catchments considered to be hydrologically similar to the local catchments. The following figures show the time versus discharge relationship for each S-graph.

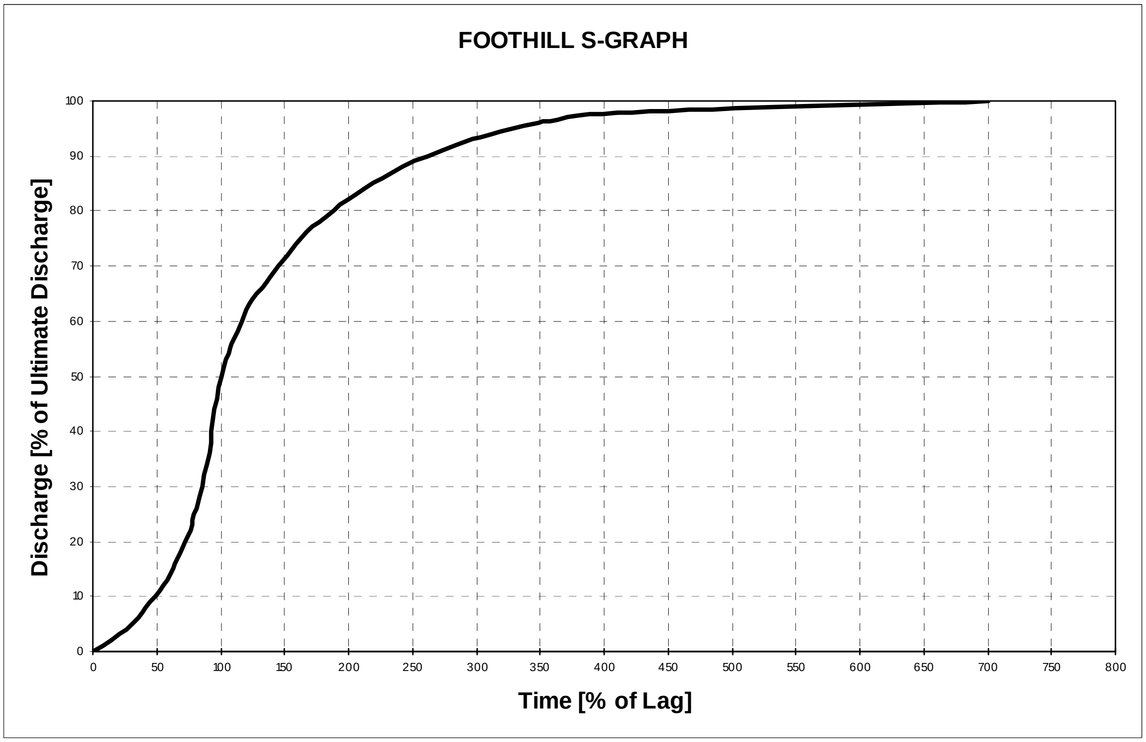


Figure 6- 3. San Joaquin County Foothill S-graph

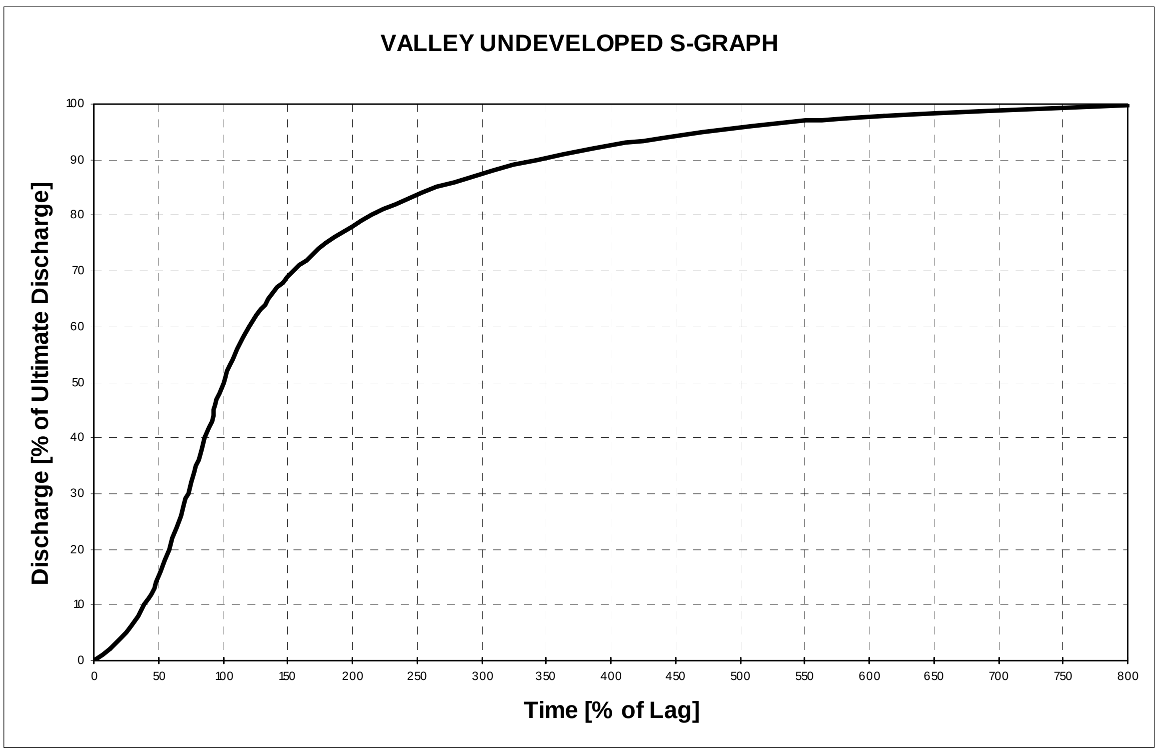


Figure 6- 4. San Joaquin County Valley Undeveloped S-graph

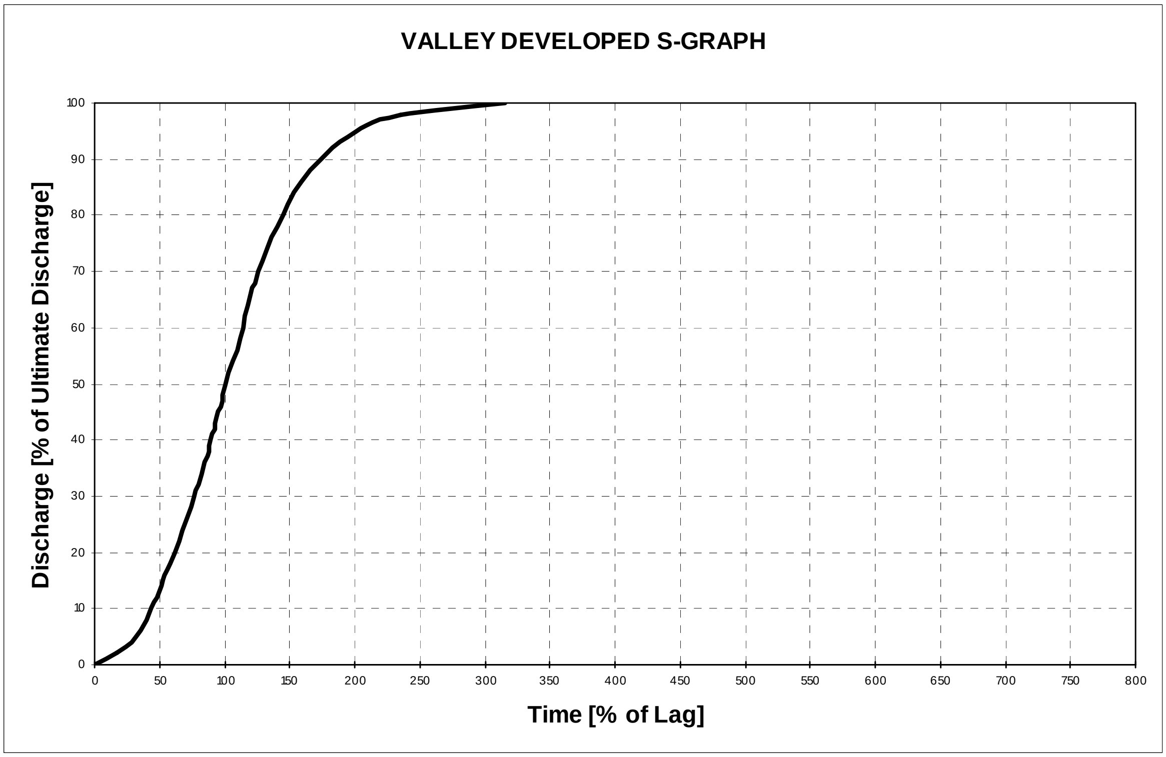


Figure 6- 5. San Joaquin County Valley Developed S-graph

Basin lag times were calculated according to guidelines set forth in the San Joaquin County Hydrology Manual[10]. The following equation was used:

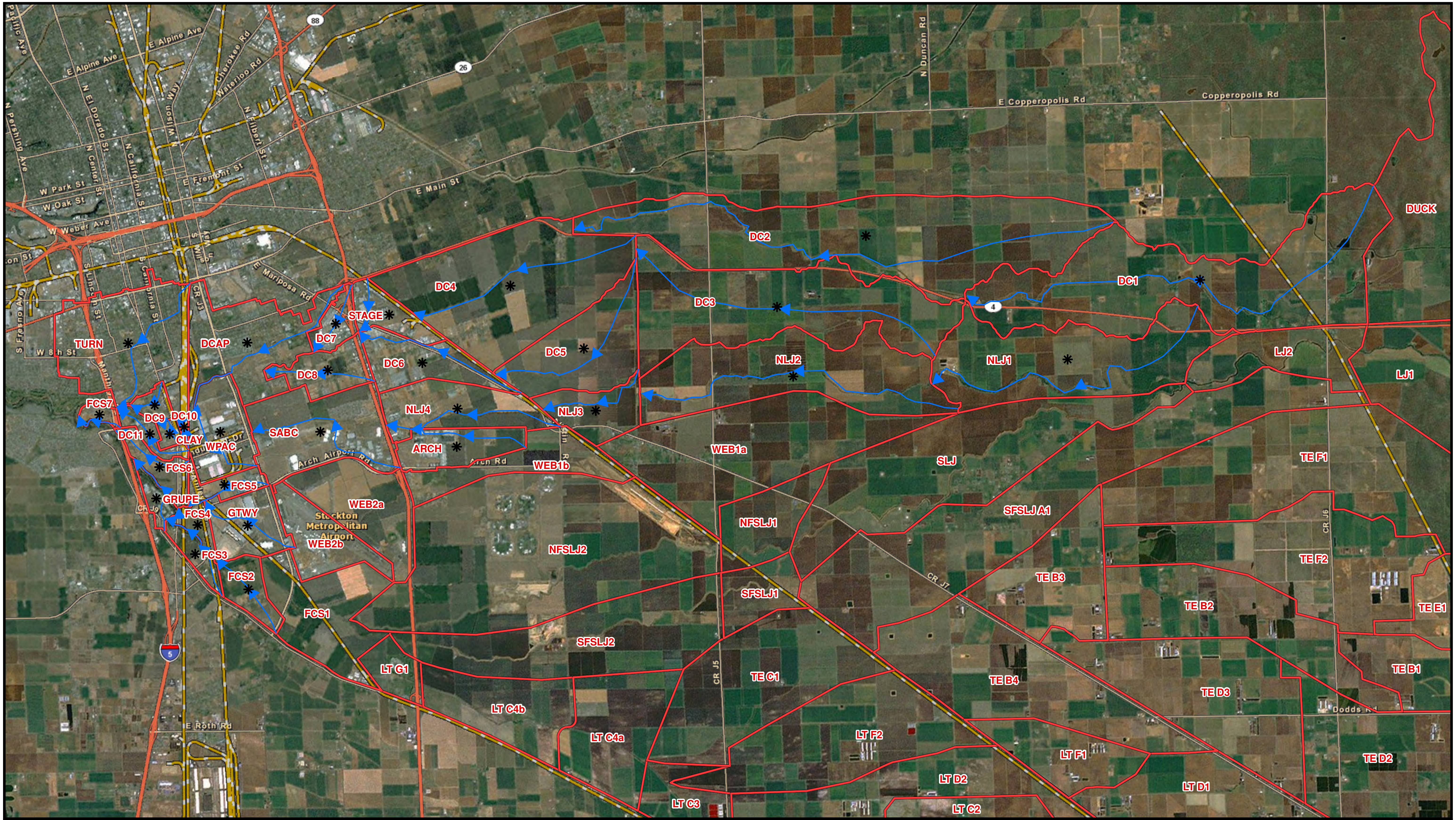
$$Lg = 24n(L \cdot L_C/S^{0.50})^{0.38}$$

where:

Lg	=	Lag time [hours]
n	=	Average basin factor estimated using Figure E-2 in the San Joaquin County Hydrology Manual
L	=	Length of longest watercourse [miles]
L_C	=	Length of longest watercourse measured to the centroid of the basin [miles]
S	=	Overall slope of longest watercourse [feet/mile]

For the 55 subbasins that originated from the Tidewater Model (see Figure 6- 2), L, L_C, S and n values were determined by Domenichelli & Associates[25].

For the remaining 30 subbasins, lag time parameters were calculated by PBI using ArcGIS software. Flowpaths identified for these calculations are shown in Figure 6- 6. S-graph assignments and lag time calculations for each subbasin are provided in Attachment 6-D and Attachment 6-E for 'Existing' and 'Future-Without-Project' Conditions, respectively.



Subbasin Boundary Subbasin Centroid Subbasin Flowpath	N	0 0.5 1 Miles 1 : 75,000 OCTOBER 19, 2010	PETERSON . BRUSTAD . INC ENGINEERING . CONSULTING 1180 Iron Point Rd., Suite 260 Folsom, CA 95630 Phone: (916) 608-2212 Fax: (916) 608-2232	SAN JOAQUIN AREA FLOOD CONTROL AGENCY FRENCH CAMP SLOUGH SUBBASIN FLOWPATHS CALCULATED BY PBI	FIGURE 6-6

6.3.6. Channel Routing

The PBI Model includes 72 routing reaches to represent attenuation of flood waves within channels. Forty-six (46) routing reaches were imported from the Tidewater HEC-HMS model. Muskingum, Muskingum-Cunge, Kinematic Wave, Lag, and Modified Puls routing methods were all implemented for these reaches depending on conditions observed in the field during development of the Tidewater model.

Ten (10) routing reaches were imported from the Mariposa Lakes HEC-1 model. These reaches use the Muskingum-Cunge routing method with channel parameters measured during development of the Mariposa Lakes model.

The remaining 16 reaches were added by PBI and used the Muskingum-Cunge routing method. Reach lengths and slopes were measured using ArcGIS software. Manning's n values were assigned based on recommendations made in the San Joaquin County Flood Insurance Study conducted by the Federal Emergency Management Agency (FEMA)[28]. Channel cross-sections were cut using detailed topographic data from DWR LiDAR dataset[4].

6.3.7. Loss Rates

Subbasins for the PBI Model utilize the initial and constant loss rate method in HEC-HMS to model subbasin losses.

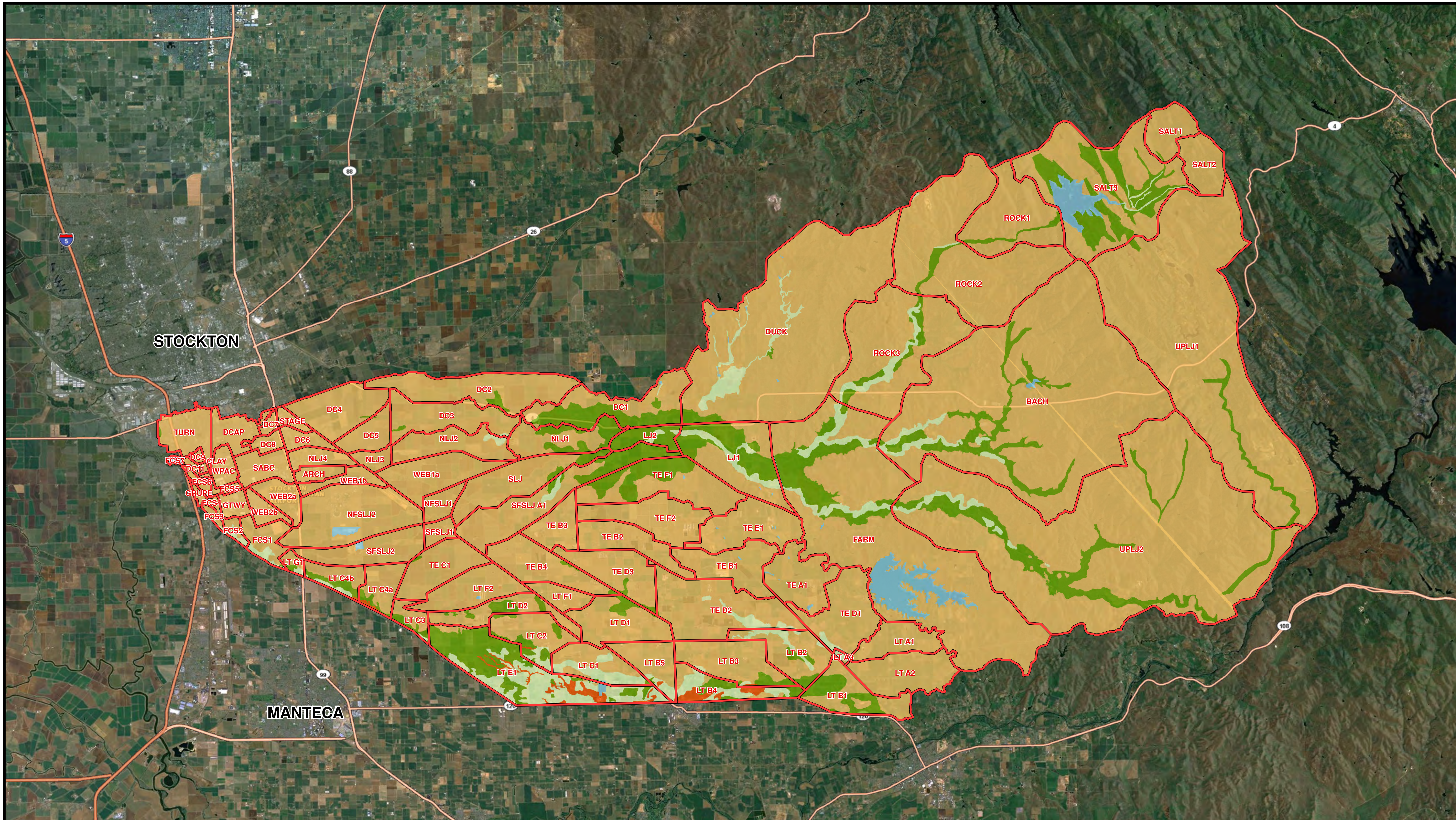
The Natural Resources Conservation Service (NRCS) has classified all soils into four hydrologic soil groups (A, B, C, and D) according to their infiltration rates[15]:

Table 6- 3. NRCS hydrologic soil groups.

Hydrologic Soil Group	Loss Rate Range [in/hr]	PBI's Assumed Loss Rate[a] [in/hr]
A	> 0.30	0.35
B	0.15 - 0.30	0.2
C	0.05 - 0.15	0.1
D	0.00 - 0.05	0.025

[a]This loss rate value was assigned to each soil group for initial calculations of composite loss rates. The calculated composite loss rates were then adjusted during the calibration process.

GIS soils data was obtained from the NRCS[13] and used to determine the proportional coverage of soil groups within French Camp Slough subbasins (Figure 6- 7). NRCS GIS soils data was not available for Calaveras County. Soils data for this part of the study area was obtained from the Calaveras County Soil-Vegetation Survey[14].



Group A	Group D	
Group B	Water/Other	
Group C		

N

0 1.5 3 Miles

1 inch = 3 miles

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FRENCH CAMP SLOUGH
SOILS MAP

FIGURE

6-7

A weighted average of loss rates was then calculated for each subbasin and adjusted during the calibration process (See Section 6.4). After the calibration adjustment, subbasin loss rates range from 0.021 inches per hour to 0.144 inches per hour as shown in Attachment 6-F.

EM 1110-2-1417[18] recommends that initial losses are set between 0.5-1.5 inches for agricultural areas. Initial losses were set to 0.5 inches for all agricultural/rural subbasins in the foothills and to 1.5 inches for agricultural/rural subbasins in the valley. For urban subbasins, initial losses were set to 0.2 inches also based on guidelines listed in *EM 1110-2-1417*.

6.3.8. Impervious Percentages

Impervious percentages were assigned based on the extent of urbanization within each subbasin. Aerial photos including those contained within 2010 LiDAR datasets[4] were used to assess existing urbanization in the French Camp Slough watershed. The impervious percentages corresponding to each land use type were selected with the guidance of the San Joaquin County *Hydrology Manual*[10].

Table 6- 4. Land use types and their corresponding impervious percentages.

Land Use Type	Impervious Percentage
Agricultural/Open Space	2%
Agricultural with Rural Residential Development	5%
Fully Developed Residential	60%
Industrial	90%

6.4. MODEL CALIBRATION

Calibration to an observed rainfall/runoff event was considered for the PBI Model, however there was very little concurrent rainfall/runoff data in the French Camp Slough watershed. The available runoff data included stage recordings and did not include a rating curve. Calibration to an observed event would have contained a large amount of uncertainty and therefore was not included in the French Camp Slough analysis.

Constant loss rates were adjusted for each subbasin by a factor of 0.85 (Attachment 6-F). The adjustment factor was determined through a HEC-HMS calibration for the neighboring Calaveras River watershed. This watershed has similar characteristics to the French Camp

Slough watershed and has more reliable stream flow data. Further details of the Calaveras River model calibration can be found in Section 5.4.

6.5. DEVELOPMENT CONDITIONS

6.5.1. Existing Conditions

An ‘Existing Conditions’ model run was performed to evaluate peak flows given current (2010) land use and hydrologic conditions within the French Camp Slough watershed. Subbasin S-graphs, ‘n’ values, and impervious percentages were set according to current land cover conditions using field knowledge supplemented by aerial photos.

In general, the upstream watershed consists of natural or agricultural land whereas the lower portions of French Camp Slough watershed (west of Highway 99) are developed areas in and around the city of Stockton. Figure 6- 8 displays the existing development conditions and S-graphs assigned to each subbasin. A summary table of the subbasin characteristics used for ‘Existing Conditions’ model runs is provided in Attachment 6-D.

6.5.2. Future-Without-Project Conditions

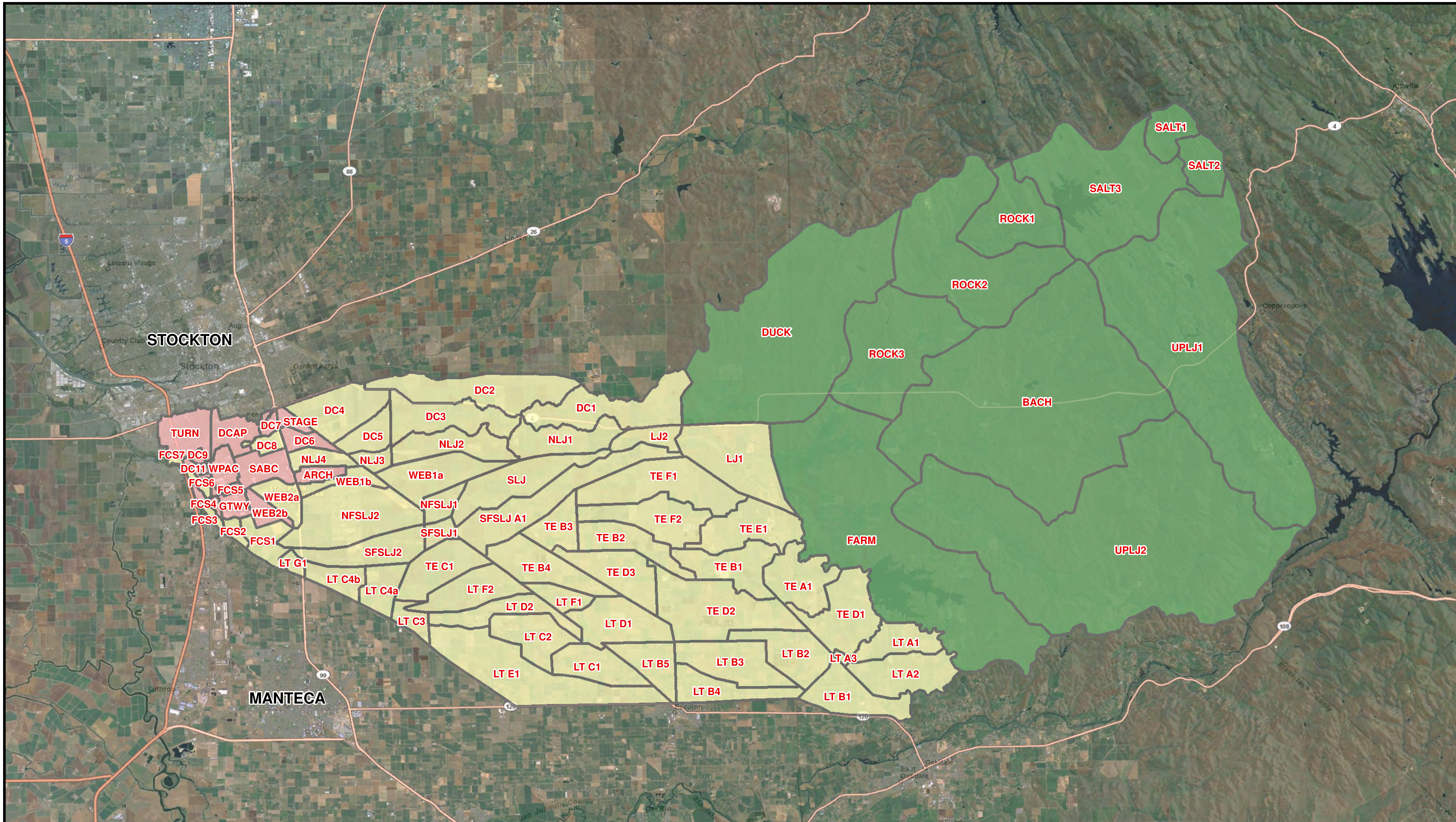
A ‘Future-Without-Project Conditions’ model run was performed to evaluate peak flows for future (2070) land use and hydrologic conditions within the French Camp Slough watershed.

Future land use conditions are based on the City of Stockton 2035 General Plan[12] and the San Joaquin County General Plan[17].

As shown in Figure 6- 9, the upstream watershed remains unchanged and consists of natural or agricultural land whereas the lower portions of French Camp Slough watershed experience an increase in development. There are sixteen (16) subbasins that were previously undeveloped in the ‘Existing Conditions’ model and are assumed to be fully developed for the ‘Future-Without-Project Conditions’ model.

In addition to updating subbasin S-graphs, ‘n’ values, and impervious percentages for the newly developed areas, storm water pump stations were also added to these subbasins. As previously mentioned, flows exceeding pump station capacities would cause temporary ponding, which was assumed to be mitigated within the subbasin through on-site detention.

A summary table of subbasin characteristics used for ‘Future-Without-Project Conditions’ model runs is provided in Attachment 6-E.



Valley Developed S-Graph

Valley Undeveloped S-Graph

Foothill S-Graph

N

0 1.5 3 Miles

1 inch = 3 miles

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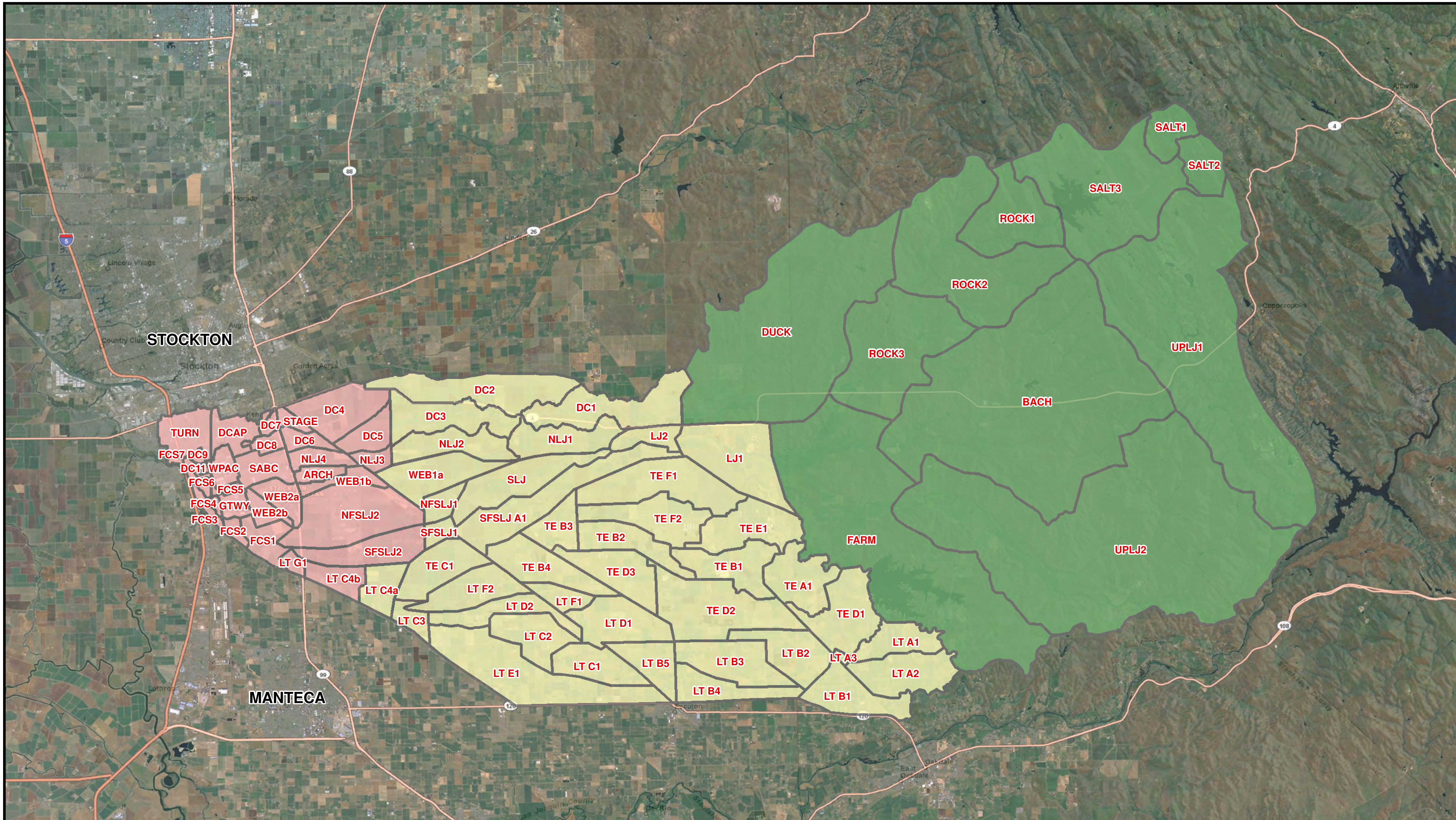
EXISTING DEVELOPMENT CONDITIONS FOR FRENCH CAMP SLOUGH WATERSHED

FIGURE

6-8



	Valley Developed S-Graph
	Valley Undeveloped S-Graph
	Foothill S-Graph

N

0 1.5 3 Miles

1 inch = 3 miles

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SAN JOAQUIN AREA FLOOD CONTROL AGENCY

FUTURE DEVELOPMENT CONDITIONS FOR FRENCH CAMP SLOUGH WATERSHED

FIGURE 6-9

6.6. STORM CENTERINGS

Four storm centerings were analyzed for the French Camp Slough watershed:

- The *Upper Watershed* centering was analyzed to stress the foothill region of the study area which could produce flash flooding.

- The *Farmington* centering was placed directly above Farmington Reservoir and was analyzed to stress Farmington Dam.

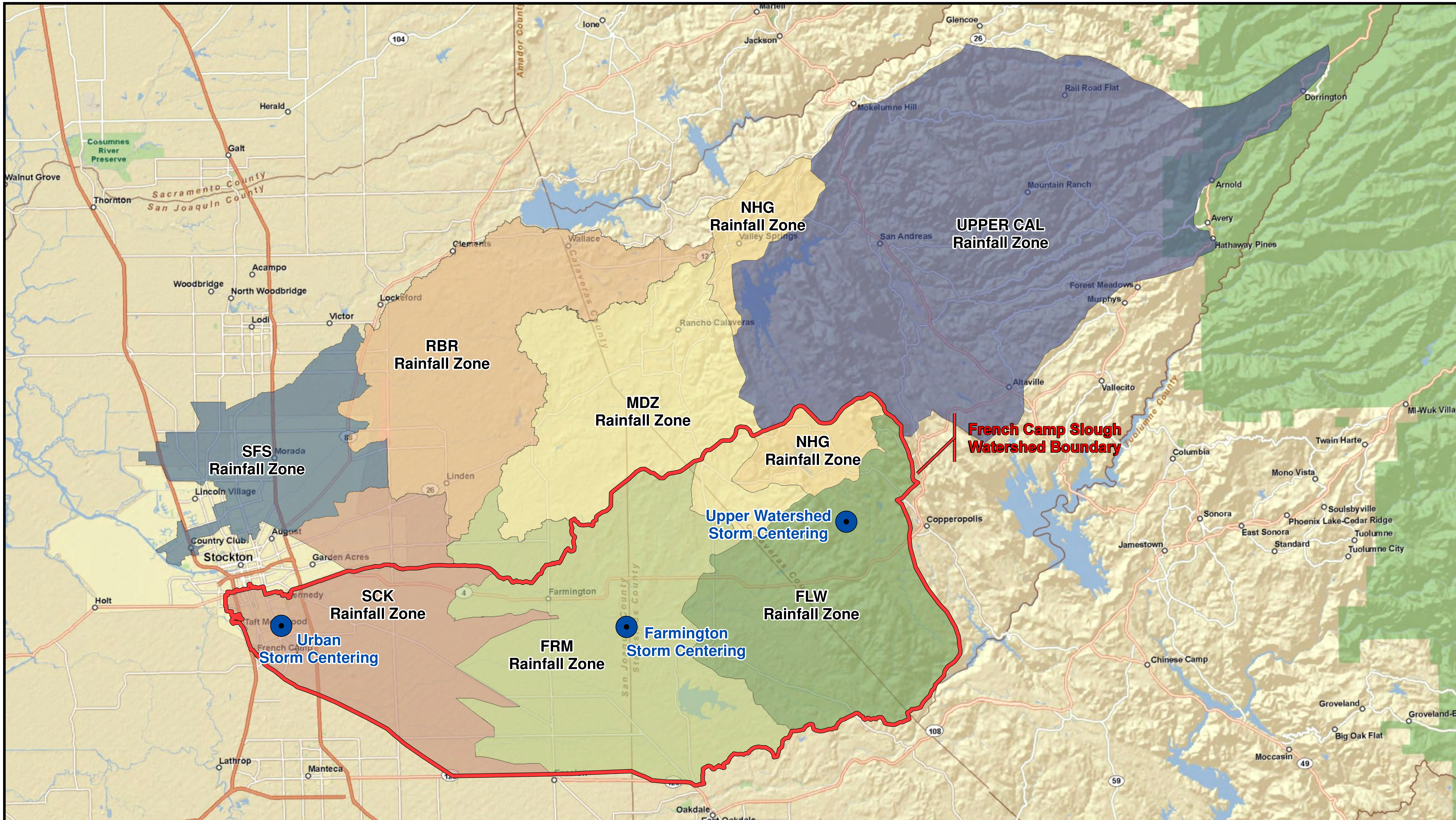
- The *"Average"* centering took the average of the *Upper Watershed* and *Farmington* area reduction factors to come up with rainfall depths. This centering is considered the official LSJRFS design storm for the French Camp Slough production runs.

  Reservoir flows from the Ford analysis were reported for an "Average" storm centering and are most applicable for this scenario. The "Average" storm centering scenario is considered the LSJRFS design storm scenario for French Camp Slough production runs and therefore are the only flows reported in Table 6- 6 and Table 6-7.

- The *Urban* centering was analyzed for interior drainage purposes and is directly centered over the urban areas in the lower watershed.

Eight design storms with the 1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, and 1/500 AEP events will be produced for each centering. This selection of design storms provides a wide range of scenarios that can be used for planning purposes.

Calculated area reduction factors and resulting area-reduced rainfall depths for each rainfall zone are provided in Attachment 6-G for all frequency-duration-storm centering combinations.



0 5 Miles 1 : 300,000 JUNE 24, 2011	PETERSON . BRUSTAD . INC ENGINEERING . CONSULTING 1180 Iron Point Rd., Suite 260 Folsom, CA 95630 Phone: (916) 608-2212 Fax: (916) 608-2232	LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY	FIGURE	
		FRENCH CAMP SLOUGH WATERSHED STORM CENTERINGS	6-10	

6.7. MODEL SIMULATIONS

French Camp Slough production runs include 64 scenarios with unique combinations of development conditions, storm frequencies, and storm centerings.

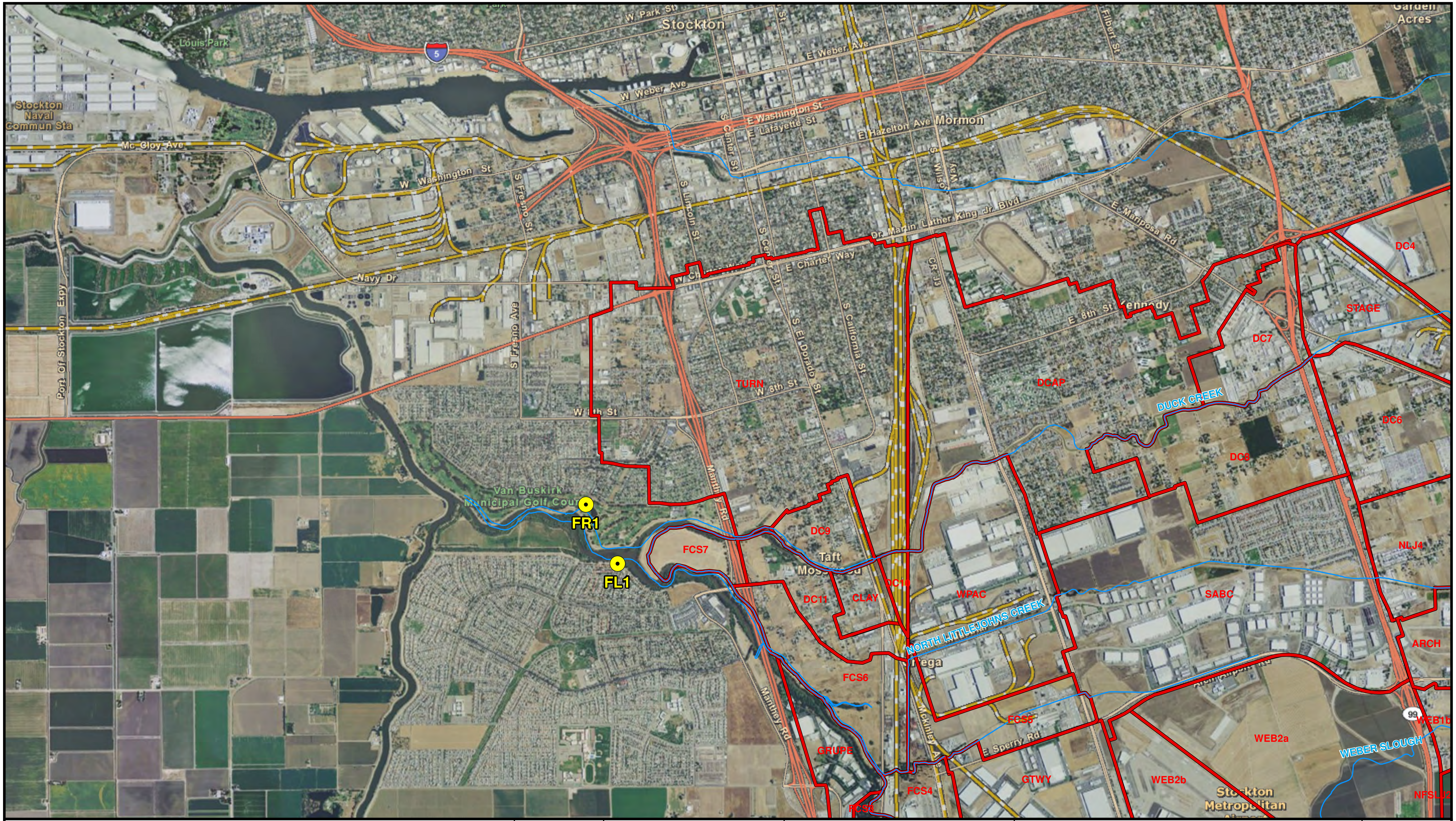
Table 6- 5. French Camp Slough production run scenarios.

Development Conditions	Storm Centerings	AEP Events
Existing Conditions	Urban	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	Farmington	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	"Average"	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	Upper Watershed	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
Future-Without-Project Conditions (2070)	Urban	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	Farmington	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	"Average"	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500
	Upper Watershed	1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, 1/500

6.7.1. Summary of Results

Peak flow results were extracted from HEC-HMS at each LSJRFS index point. Locations of LSJRFS index points within the French Camp Slough watershed are shown in Figure 6- 11. Table 6- 6 and Table 6-8 summarize “Average” storm centering peak flows for ‘Existing Conditions’ runs and for ‘Future-Without-Project Conditions’ runs, respectively.

Reservoir flows from the Ford analysis were reported for an “Average” storm centering and are most applicable for this scenario. The “Average” storm centering scenario is considered the LSJRFS design storm scenario for the Calaveras River production runs and therefore are the only flows reported in Table 6- 6 and Table 6-8.



Subshed Boundary	
LSJRFS Index Point	

0 0.125 0.25 0.5 Miles

1 inch = 1/2 mile

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LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

FRENCH CAMP SLOUGH WATERSHED INDEX POINTS

FIGURE

6-11

Table 6-6. Peak Flow Results for French Camp Slough - Existing Conditions [cfs]

LSJRFS Index Point ID	Description	Average Storm Centering							
		1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP
--	Littlejohns Creek at Town of Farmington[1]	1,400	2,170	2,370	2,620	3,740	9,900	12,900	16,600
--	Duck Creek at Hwy 99	410	500	740	1,050	1,310	1,570	1,800	2,140
--	North Littlejohns Creek at Hwy 99	20	50	130	250	360	460	560	750
--	North Fork- South LJ Creek at Hwy 99	650	1,010	1,110	1,200	1,340	1,700	1,910	1,970
--	South Fork- South LJ Creek at Hwy 99	760	1,190	1,350	1,500	1,890	3,860	5,900	7,730
--	FCS at UPRR	1,440	2,540	2,860	3,170	3,590	5,030	6,070	7,020
D7 & D8	FCS at Duck Creek Confluence	1,790	3,030	3,860	4,710	5,500	6,490	7,090	7,800

[1]Flows for Littlejohns Creek at Town of Farmington were Provided by USACE. All upstream model flows were superseded by USACE hydrographs at Farmington.

Table 6-7. Peak Flow Results for French Camp Slough - Future Conditions [cfs]

LSJRFS Index Point ID	Description	Average Storm Centering							
		1/2 AEP	1/5 AEP	1/10 AEP	1/25 AEP	1/50 AEP	1/100 AEP	1/200 AEP	1/500 AEP
--	Littlejohns Creek at Town of Farmington[1]	1,400	2,170	2,370	2,620	3,740	9,900	12,900	16,600
--	Duck Creek at Hwy 99	1,140	1,470	1,530	1,590	1,740	1,770	1,790	1,990
--	North Littlejohns Creek at Hwy 99	240	300	340	470	470	480	550	750
--	North Fork- South LJ Creek at Hwy 99	720	1,090	1,200	1,300	1,390	1,730	1,920	1,980
--	South Fork- South LJ Creek at Hwy 99	820	1,270	1,450	1,590	1,900	3,850	5,890	7,720
--	FCS at UPRR	1,590	2,700	3,030	3,340	3,730	5,060	6,090	7,030
D7 & D8	FCS at Duck Creek Confluence	2,050	3,290	4,060	4,870	5,660	6,610	7,200	7,840

[1]Flows for Littlejohns Creek at Town of Farmington were Provided by USACE. All upstream model flows were superseded by USACE hydrographs at Farmington.

6.7.2. Uncertainty Parameters

For the purposes of the LSJRFS, uncertainty parameters for each flow-frequency dataset can be estimated within HEC-FDA during the project's economic analysis. HEC-FDA defines uncertainty in terms of confidence intervals or standard deviations given inputs of flow-frequency data (provided in Table 6- 6 and Table 6-8) and an equivalent record length.

The equivalent record length is an estimate of the overall "worth" or "quality" of the flow-frequency function, expressed as the number of years-of-record[19]. For probability functions derived at ungaged locations using model or other data, the equivalent record length is based on a judgment of the quality of that model or data. EM 1110-2-1619[20] provides guidelines for assigning equivalent record lengths.

The French Camp Slough model wasn't calibrated to an observed event; however the parameters were adjusted based on the calibration of the neighboring Calaveras River model (see Section 5.4). Because of this, there is slightly less confidence in the French Camp flows compared to flows calculated for the other LSJRFS watersheds. EM 1619 estimates that the equivalent record length is 10-30 years for a rainfall-runoff model with *regional* adjustments made to its parameters.

7.0 REFERENCES

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8.0 ATTACHMENTS

Attachment 2- A. Procedure for Calculating Area Reduction Factors: USACE Guadalupe River Hydrology Report[2]

Documentation of Meteorology Calculations

The purpose of this document is to outline how the 3-day statistical precipitation patterns for the sub-basins were calculated. The following will go through the calculations performed in the Excel spreadsheet created by the Sacramento District which linearly interpolates between the Depth Area Reduction Factors (DARFs) as presented in HMR 59 for various drainage areas, computes the statistical 1, 6, 12, 24, 48, and 72-hour cumulative precipitation for each sub-basin, and creates custom hyetographs.

Depth Area Reduction Factors (DARFs)

	A	B	C	D	E	F	G	H	I	J	K	L	M
1	CALIFORNIA AREA 3 MIDCOAST MTN, CA --UPDATED MARCH 2006												
2	CHECK NEEDED												
3	HMR RATIOS AND DAD CURVES FROM HMR 58												
4	CALIFORNIA AREA 3 UPDATED Augest 1999 BY RBC												
5		MIDCOAST MTN, CA				ALL SEASON							
6	HRS	1		6		12		24		48		72	
7	24RATIO	0.130	0.2	0.450	0.49	0.740	0.7	1.000	1.1	1.450	1.6	1.700	0
8	DA												
9	0.1	1	0	1	0	1	0	1	0	1	0	1	0
10	10	1	-0.00313	1	-0.00281	1	-0.00250	1	-0.00225	1	-0.00200	1	-0.00175
11	50	0.875	-0.00115	0.888	-0.00100	0.900	-0.00090	0.910	-0.00080	0.920	-0.00070	0.930	-0.00060
12	100	0.818	-0.00060	0.838	-0.00055	0.855	-0.00050	0.870	-0.00045	0.885	-0.00040	0.900	-0.00038
13	200	0.758	-0.00028	0.783	-0.00024	0.805	-0.00023	0.825	-0.00022	0.845	-0.00020	0.863	-0.00019
14	500	0.675	-0.00014	0.710	-0.00011	0.735	-0.00011	0.760	-0.00011	0.785	-0.00011	0.805	-0.00010
15	1000	0.608	-0.00008	0.655	-0.00007	0.680	-0.00007	0.705	-0.00006	0.730	-0.00006	0.755	-0.00006
16	2000	0.530	-0.00005	0.585	-0.00005	0.615	-0.00004	0.640	-0.00004	0.670	-0.00004	0.700	-0.00004
17	5000	0.380	-0.00003	0.445	-0.00002	0.485	-0.00002	0.520	-0.00002	0.550	-0.00002	0.590	-0.00002
18	10000	0.250	0	0.340	0	0.380	0	0.420	0	0.450	0	0.490	0

The Depth Area Reduction Factors (DARFs) from HMR 59 are represented in the Excel spreadsheet as shown above. Columns B, D, F, H, J, and L contain the DARFs for the 1, 6, 12, 24, 48, and 72-hour durations, respectively. The columns in-between (Columns C, E, G, I, K, and M) calculate the incremental change in DARF for a given unit of area (1 square mile). These numbers are used to linearly interpolate between DARF values for drainage areas in-between those specified in column A.

The row labeled “24RATIO” (Row 7) refers to the ratio to 24-hour cumulative precipitation as derived in HMR 59. These ratios are used to convert the 24-hour precipitation (main input parameter) to 1, 6, 12, 48, and 72-hour precipitation for each sub-basin.

For the purpose of this study, the California Area 3 Midcoast Mountain, California DARFs were selected, as noted in Row 1.

Cumulative Precipitation Calculations

	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI	AJ	AK	AL	AM
1																	
2	Curve					ENTER	Enter data as indicated below										
3	No.	HMR 58 SIERRA MTN SUBAREA 5	Month diff from all-season (0 to 5)-->			0	<-----	Enter data here									
4	Use	TO USE	Off season month ---> (Name, e.g. APR			All Season	<-----	Enter data here									
5	0-5		Seasonal Ratio from fig 13.X, or 2			0	<-----	Enter Basion Ratio or enter 2 for Vering basion ratios									
6	0	All season						Other data entry columns are shaded Blue									
7	1	Offset +/- 1 Month For April or October		HMR 58 SIERRA MTN SUBAREA 5													
8	2	Offset +/-2 Months For May Or September		Guadalupe River				If the off season month subbasin ratios very from basin to basin enter in this column.									
9	3	Offset +/- 3 Months For June Or August		All Season													
10	4	Offset +/- 4 Month July		All season					If all subbasins have the same ratio value in cell 'AB5' is used.								
11	5	Offset + or - 5 months None in Sierra Mtn. Regon		Upper Guadalupe Centering													
12	6	Multi Region Curves															
13							24-HR			24-HR		24-HR	24-HR	24-HR	1-HR	1-HR	1-HR
14	RANK			Station ID		Drainage		Off-	Off-		Accum-	DA*PPT=	Accum-		Ppt	Ppt	Ppt
15	NO.					Area	Max	Season	Season	Max	ulated	SINGLE	ulated	Average	specific		Adjusted
16		Guadalupe River					Ppt	Variable	Month	Ppt	Drainage	BASIN	BASIN	Depth,	DA		concurrent
17		Upper Guadalupe Centering						Ratio	1 for all		Area (DA)	VOLUME	VOLUME	Accum-			DA
18									season					ulated	Ratio for 1HR		
19								1 for all	1					BASIN	1		
20								season						VOLUME	0.130		
21															Adjusted for DA		
22				A=Part	B=PART												
23		Sub-Basin Description				sq.mi.	in.			in.	sq.mi.				in.	in.	in.
24	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
25																	
26	1	Upstream from Guadalupe Reservoir		W1730		4.3	9.58	1	1.00	9.58	4.3	41.50	41.50	9.58	1.24	1.24	1.24
27	2	Upstream from Guadalupe Reservoir		W1970		1.6	9.47	1	1.00	9.47	6.0	15.32	56.82	9.55	1.23	1.24	1.23
28	3	Alamitos Creek		W1480		2.3	7.00	1	1.00	7.00	8.3	16.10	72.92	8.84	0.91	1.15	0.91
29	4	Upstream from Almaden Reservoir		W1850		11.9	9.87	1	1.00	9.87	20.1	117.11	190.03	9.45	1.28	1.19	1.22
30	5	Downstream from Almaden Reservoir		W1540		4.3	7.16	1	1.00	7.16	24.5	31.07	221.10	9.04	0.93	1.12	0.81
31	6	Upstream from Lake Elsman		W1950		9.7	11.47	1	1.00	11.47	34.2	111.80	332.90	9.73	1.49	1.17	1.29
32	7	Upstream Alamden Lake Alamitos Creek		W1400		6.5	7.42	1	1.00	7.42	40.7	48.46	381.36	9.36	0.96	1.10	0.74
33	8	Downstream from Guadalupe Reservoir		W1960		6.7	9.28	1	1.00	9.28	47.5	62.50	443.86	9.35	1.21	1.07	0.91
34	9	Upstream from Lexington Reservoir		W1800		2.4	9.69	1	1.00	9.69	49.9	23.47	467.33	9.37	1.26	1.07	0.92
35	10	Below Almaden Res		Below Almaden		1.7	6.62	1	1.00	6.62	51.6	11.05	478.38	9.28	0.86	1.05	0.67

Fields shaded in light blue represent cells in which user input is possible or required.

Row 3, Column AB is populated with a “0” to represent “all-season” conditions. Seasonal variations in precipitation patterns are not accounted for in this study.

Row 26 and below:

- Column W is populated with the sub-basin “rank number”. Each sub-basin is assigned a rank, starting with the storm center and ending at the mouth of the basin.
- Column X is populated with a description of each sub-basin.

3

- Column Z includes the sub-basin ID, as used in the HMS model.
- Column AB includes the individual drainage area of each sub-basin.
- Column AC is populated with the calculated 24-hour cumulative precipitation for each sub-basin given a particular event frequency (i.e. 2,5,10,25, 50, 100, 200 or 500-year).
- Columns AD and AE are set at 1, for "all seasons."
- The remaining Columns (not shaded in light blue) conduct various calculations on the inputted data.
    - Column AF remains the same as Column AC for "all seasons."
    - Column AG calculates the cumulative drainage area. This is done by summing up all of the drainage areas at a given sub-basin (rank x) from rank 1 to rank x. This number is used to estimate the drainage area upstream of a particular sub-basin. This number is used in selecting an appropriate set of DARFs for a given sub-basin. For example, for sub-basin W1850 with a "rank number" of 4, the cumulative drainage area is 20.1. The DARFs are then calculated using the table previously presented using a linear interpolation between the DARFs for 10 square miles and the DARFs for 50 square miles.
    - Column AH calculates the volume of water allotted to a given sub-basin during the maximum 24-hour duration.
    - Column AI calculates the cumulative volume of water at a given sub-basin (rank x) from rank 1 to rank x. This is used to make sure that the volume of water over the entire basin is conserved.
    - Column AJ calculates the average depth of water for the cumulative drainage area above a given sub-basin. This is calculated by dividing the cumulative water volume (Column AI) by the cumulative drainage area (Column AG). This number is used in subsequent calculations to estimate the specific and concurrent precipitation at a given index location (at the outlet of a given sub-basin) with respect to the entire drainage area above the index location.
    - Columns AK through AM conduct calculations for the 1-hour duration that are subsequently carried out for the remaining durations (6, 12, 24, 48, and 72-hours) to estimate the "specific" and "concurrent" precipitation:
        - Column AK uses the ratio to 24-hour precipitation, 0.130 (see previous table), to estimate the maximum cumulative 1-hour specific precipitation.

- Column AL represents an intermediary calculation necessary to calculate the concurrent precipitation. This field multiplies the average depth for the cumulative drainage area above a given sub-basin (Column AJ) by the ratio to 24-hour precipitation (0.130), and applies the appropriate DARF using the aforementioned table.

- Column AM calculates the concurrent cumulative precipitation by subtracting the product of the accumulated drainage area (Column AG) and Column AL from the previous sub-basin (by rank) from the product of Column AG and Column AL of the given sub-basin, and dividing this difference by the drainage area of the given sub-basin. This calculates the precipitation depth for a particular sub-basin while taking into account its placement in the larger basin area with respect to the storm center. In other words:

  *PptCon10 = (CumDA10*PptSp10 – CumDAR9*PptSp9)/DA10*

  PptCon10 = Concurrent cumulative precipitation for a sub-basin of a given rank (say, rank = 10).
  CumDA10 = Cumulative Drainage Area of the sub-basin (rank = 10)
  PptSp10 = Specific cumulative precipitation for the sub-basin, previously calculated using the average rainfall depth over all of the previous sub-basins (ranks 1 through 10).
  CumDAR9 = Cumulative drainage area for the previous sub-basin (rank = 9).
  PptSp9 = Specific cumulative precipitation for the previous sub-basin (rank = 9), previously calculated.
  DA10 = Individual drainage area for the sub-basin (rank = 10).

5

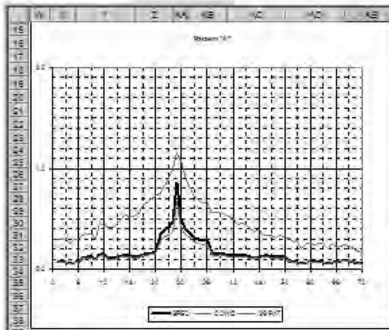
6-Hour Average Precipitation

	BG	BH	BI	BJ	BK
49	Below Almaden Res				
50		HMR59		HMR59	
51	PERIOD	Specific	AVERAGE	Concurrent	AVERAGE
52		Centering	6-HR	centering	6-HR
53			PERIODS		PERIODS
54	1-HR	0.86	5.16	0.67	4.04
55	6HR-1HR	2.12		1.73	
56	6-HR	2.98	2.98	2.40	2.40
57	12HR-6HR	1.92	1.92	1.65	1.65
58	24HR-12HR	1.72	0.86	1.54	0.77
59	48HR-24HR	2.98	0.74	2.69	0.67
60	72HR-48HR	1.65	0.41	1.63	0.41

These calculations will be used in distributing the calculated 1, 6, 12, 24, 48, and 72-hour specific and concurrent rainfall over the chosen precipitation pattern (“Pattern A”).

- Columns BH and BJ present the previously calculated specific and concurrent rainfall depths, and various differences between depths.

- Columns BI and BK calculate the average 6-hour rainfall of a given period (when applicable, no 6-hour average calculated for Row 55).



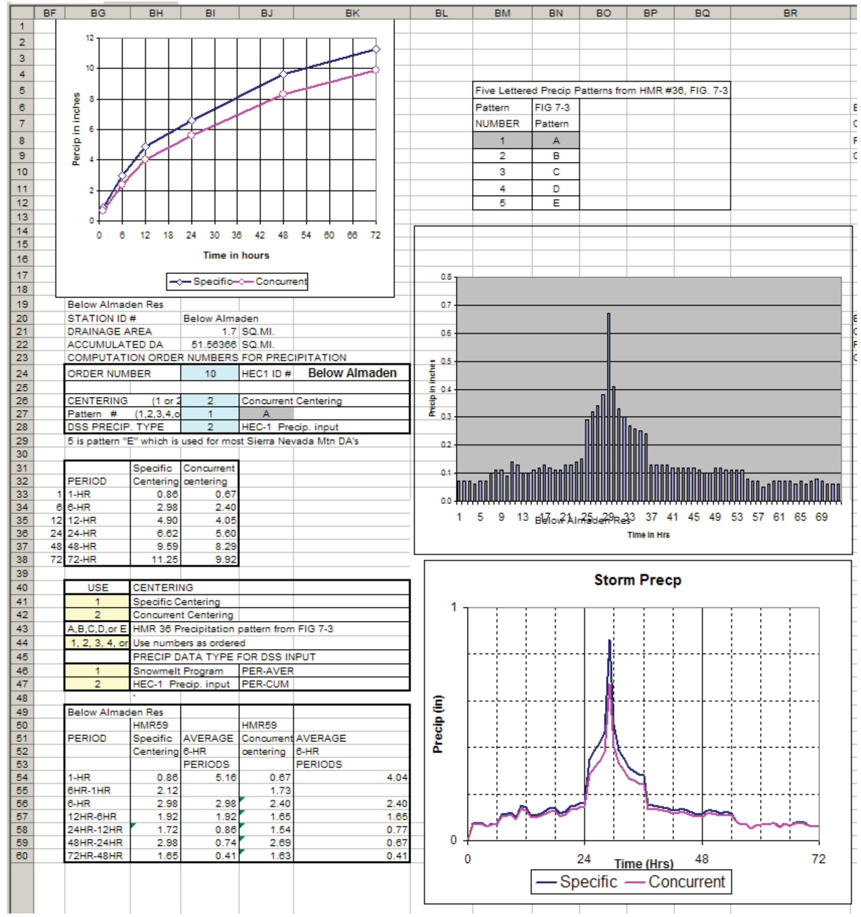
	W	X	Y	Z	AA	AB PERIOD COUNT	AC HMR52 PMP PATTERN	AD BASIN PATTERN HMR59 Specific Centering	AE PATTERN HMR59 Concurrent Centering
43		HMR# 36 PATTERN "A" + adjustments							
46		1	specific	0.41	72	1	0.30	0.071	0.070
47		2	conc	0.41	72	2	0.30	0.071	0.070
48	9	3	pattern total	1.74	72	3	0.30	0.071	0.070
49		4	Specific sum	0.41	72	4	0.25	0.059	0.059
50		5	concur.sum	0.41	72	5	0.30	0.071	0.070
51		6			72	6	0.29	0.069	0.068
52		1	specific	0.74	48	7	0.34	0.114	0.103
53		2	conc	0.67	48	8	0.35	0.117	0.106
54	7	3	pattern total	2.22	48	9	0.35	0.117	0.106
55		4	Specific sum	0.74	48	10	0.30	0.101	0.091
56		5	concur.sum	0.67	48	11	0.45	0.151	0.136
57		6			48	12	0.43	0.144	0.130
58		1	specific	0.74	48	13	0.42	0.109	0.099
59		2	conc	0.67	48	14	0.42	0.109	0.099
60	5	3	pattern total	2.85	48	15	0.45	0.117	0.106
61		4	Specific sum	0.74	48	16	0.50	0.130	0.118
62		5	concur.sum	0.67	48	17	0.54	0.141	0.127
63		6			48	18	0.53	0.138	0.125
64		1	specific	0.86	24	19	0.52	0.118	0.106
65		2	conc	0.77	24	20	0.55	0.124	0.112
66	3	3	pattern total	3.60	24	21	0.65	0.147	0.132
67		4	Specific sum	0.86	24	22	0.66	0.149	0.134
68		5	concur.sum	0.77	24	23	0.70	0.158	0.142
69		6			24	24	0.72	0.163	0.146
70		1	specific	2.96	6	25	0.75	0.349	0.285
71		2	conc	2.40	6	26	0.83	0.386	0.318
72	1	3	pattern total	5.70	6	27	0.90	0.419	0.342
73		4	Specific sum	2.96	6	28	1.00	0.465	0.380
74		5	concur.sum	2.40	6	29	1.15	0.860	0.674
75		6			6	30	1.07	0.498	0.407
76		1	specific	1.92	12	31	0.89	0.386	0.332
77		2	conc	1.65	12	32	0.81	0.352	0.302
78	2	3	pattern total	4.42	12	33	0.72	0.313	0.269
79		4	Specific sum	1.92	12	34	0.69	0.300	0.258
80		5	concur.sum	1.65	12	35	0.66	0.287	0.246
81		6			12	36	0.65	0.282	0.243
82		1	specific	0.86	24	37	0.57	0.149	0.134
83		2	conc	0.77	24	38	0.57	0.149	0.134
84	4	3	pattern total	3.23	24	39	0.56	0.147	0.132
85		4	Specific sum	0.86	24	40	0.55	0.144	0.129
86		5	concur.sum	0.77	24	41	0.53	0.139	0.125
87		6			24	42	0.50	0.131	0.118
88		1	specific	0.74	48	43	0.44	0.130	0.118
89		2	conc	0.67	48	44	0.46	0.136	0.123
90	6	3	pattern total	2.51	48	45	0.44	0.130	0.118
91		4	Specific sum	0.74	48	46	0.40	0.119	0.107
92		5	concur.sum	0.67	48	47	0.38	0.113	[illegible]
93		6			48	48	0.39	0.116	0.104
94		1	specific	0.74	48	49	0.34	0.129	0.117
95		2	conc	0.67	48	50	0.34	0.129	0.117
96	8	3	pattern total	1.96	48	51	0.33	0.125	0.113
97		4	Specific sum	0.74	48	52	0.31	0.118	0.106
98		5	concur.sum	0.67	48	53	0.33	0.125	0.113
99		6			48	54	0.31	0.116	0.106
100		1	specific	0.41	72	55	0.31	0.082	0.080
101		2	conc	0.41	72	56	0.27	0.071	0.070
102	9	3	pattern total	1.57	72	57	0.28	0.074	0.073
103		4	Specific sum	0.41	72	58	0.20	0.053	0.052
104		5	concur.sum	0.41	72	59	0.25	0.066	0.065
105		6			72	60	0.26	0.068	0.067
106		1	specific	0.41	72	61	0.24	0.070	0.069
107		2	conc	0.41	72	62	0.25	0.073	0.072
108	5	3	pattern total	1.41	72	63	0.25	0.073	0.072
109		4	Specific sum	0.41	72	64	0.20	0.059	0.058
110		5	concur.sum	0.41	72	65	0.25	0.073	0.072
111		6			72	66	0.22	0.065	0.064
112		1	specific	0.41	72	67	0.23	0.075	0.074
113		2	conc	0.41	72	68	0.24	0.079	0.078
114	12	3	pattern total	1.26	72	69	0.23	0.075	0.074
115		4	Specific sum	0.41	72	70	0.20	0.066	0.065
116		5	concur.sum	0.41	72	71	0.18	0.059	0.058
117		6			72	72	0.18	0.059	0.058
119							32.73	11.25	9.92

The specific and concurrent precipitation depths for the 1, 6, 12, 24, 48, and 72-hour durations are used to create a unique rainfall pattern for each sub-basin based on a selected pattern. For the purpose of this study, “Pattern A” from HMR 59 was selected, as the critical 1, 12, and 24-hour cumulative precipitation is distributed in a fashion similar to large storms in and around the study area. All of the patterns presented in HMR 59 were taken from recorded storm events throughout California.

The calculations presented in this table distribute the specific and concurrent precipitation for the 1, 6, 12, 24, 48, and 72-hour durations while maintaining the shape of the selected pattern. The rainfall pattern (Pattern A) is broken up into 6-hour increments; the 6-hour average precipitation values previously calculated for various durations are used to estimate the precipitation for each time-step.

- Specific and concurrent rainfall is distributed based average precipitation estimated for a given 6-hour increment. Within a given 6-hour increment, Column Z contains several calculations:
    - Specific and Concurrent: pulls numbers from the 6-Hour Average Precipitation Table (presented previously) depending on which duration is represented in a given 6-hour time increment.
    - Pattern Total, Specific Sum, and Concurrent Sum: Cumulative rainfall for a given 6-hour increment from Columns AC, AD, and AE, respectively.
- Column AA signifies which data from the “6-hour precipitation” table will be used in the calculations for a given 6-hour increment. For example:
    - The beginning of the rainfall pattern (Rows 46 and 51) is characterized by the rainfall between the maximum 48-hour and 72-hour precipitation pattern. As a result, the values in Row 60 of the “6-hour precipitation” table (72HR-48HR) are used to estimate the average “specific” and “concurrent” precipitation.
    - The portion of the hyetograph between Rows 70-75 represents the maximum 6-hour pattern. As a result, the values in Row 56 of the “6-hour precipitation” table (6-HR) are used to estimate the average specific and concurrent precipitation.
- Column AC presents the selected hyetograph (Pattern A).
- Column AD displays the calculated specific hyetograph for a given sub-basin.
- Column AE displays the calculated concurrent hydrograph for a given sub-basin.
- The hyetograph displayed in Column AE is the primary output from this spreadsheet. This hyetograph can be copied and pasted directly into the meteorologic model in HMS for a particular sub-basin. In this example the output for sub-basin “Below Almaden”, rank = 10, is displayed.

Spreadsheet Output



The spreadsheet calculates the specific and concurrent hyetographs for a given sub-basin by specifying a particular “order number” or “rank” in Column BI, Row 24.

Concurrent centering is selected by placing the number “2” in Column BI, Row 26 (“1” is for specific centering only).

The rainfall pattern (Pattern A, B, C, D, or E) is selected by specifying 1, 2, 3, 4, or 5 (respectively) in Column BI, Row 27.

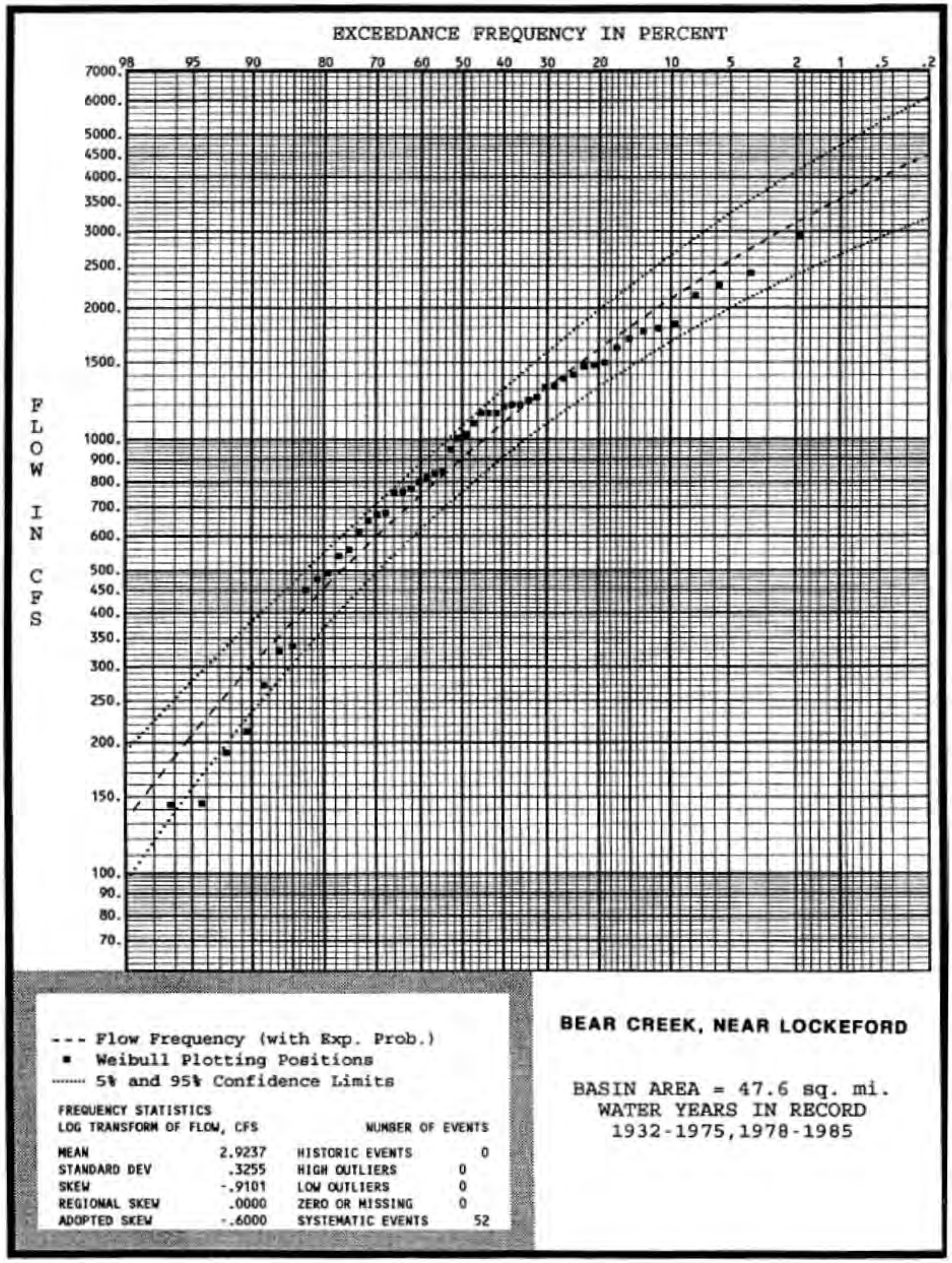
Once a desired hyetograph has been created, the spreadsheet is able to conduct various graphical representations and comparisons of the specific and concurrent precipitation patterns.

Attachment 3- A. Bear Creek Watershed Comparison of Subbasin Parameters: 1998 SJAFCA HEC-1 Model vs. 2010 PBI HEC-HMS Model

Subbasin	AREA [sq. mi] 1998 SJAFCA HEC-1 Model	AREA [sq. mi] 2010 PBI HEC-HMS Model	AREA [sq. mi] % Difference	LAG TIME [hrs] 1998 SJAFCA HEC-1 Model	LAG TIME [hrs] 2010 PBI HEC-HMS Model	LAG TIME [hrs] % Difference
B7	26.7	30.24	13%	7.80	12.59	61%
B6	13.5	11.73	-13%	5.06	9.29	84%
B5	5.6	4.04	-28%	2.97	5.83	96%
B4	2.6	1.53	-41%	2.60	4.47	72%
B10	11.7	12.01	3%	5.08	5.97	18%
B12	4.6	4.41	-4%	5.42	7.17	32%
B11	3.2	2.60	-19%	4.36	4.43	2%
B9	3.1	3.88	25%	4.84	8.73	80%
B8	1.4	0.95	-32%	3.31	3.70	12%
B13	4.71	5.28	12%	5.54	6.34	14%
B3	2.3	2.84	24%	4.76	6.29	32%
B2	1.71	2.06	21%	3.17	5.11	61%
B1	1.4	2.30	64%	2.32	3.78	63%
M3	5.67	5.76	2%	6.10	6.55	7%
M2	1.45	1.02	-29%	3.37	4.26	27%
M1	2.85	3.54	24%	4.58	4.42	-4%
LB15	0.5	0.35	-30%	0.50	0.20	-61%
LB10	1.21	0.54	-55%	0.54	0.20	-63%
LB20	1.14	0.83	-27%	0.55	0.33	-39%
LB30	0.86	0.50	-41%	0.65	0.24	-63%
LB35	1.25	0.85	-32%	0.90	0.23	-74%
LB40	1.15	1.69	47%	0.62	0.25	-60%
PX1	5	7.46	49%	6.10	5.87	-4%
LP10	1.92	1.25	-35%	2.72	2.65	-3%
LP20	1.3	0.82	-37%	2.61	2.92	12%
LP30	2.06	2.09	1%	3.57	0.41	-88%
LP31	0.79	1.10	39%	4.70	0.49	-90%
LP34[a]	--	1.25	--	--	0.32	--
LP32	1.02	0.40	-61%	1.80	0.22	-88%
LP33[a]	--	0.32	--	--	0.21	--
LB50	0.78	1.54	97%	0.48	0.34	-30%
LB55	0.06	0.28	361%	0.39	0.21	-47%
LB60	0.77	0.57	-26%	0.29	0.22	-23%
LB70	0.2	0.26	32%	0.22	0.16	-29%

[a]Subbasin parameters for LP33 and LP34 are not listed in 1998 SJAFCA HEC-1 documentation.

Attachment 3- B. Flow-Frequency for Bear Creek at Lockeford Stream Gage Used in 1998 HEC-1 Calibration



Source: HDR, Final Technical Memorandum #1, Hydrologic Report prepared for San Joaquin Area Flood Control Agency, January 1998.

Attachment 3- C. Bear Creek Subbasin Soil Groups and Loss Rates

Soil Group:	A (0.35 in/hr)	B (0.2 in/hr)	C (0.1 in/hr)	D (0.025 in/hr)	Composite Loss Rate	Adjusted Loss Rate (Adjustment Factor = 0.80)
Subbasin	[sq. mi.]	[sq. mi.]	[sq. mi.]	[sq. mi.]	[in/hr]	[in/hr]
B7	0.07	6.18	3.13	20.44	0.070	0.056
B6	0.17	1.58	2.06	7.83	0.067	0.053
B5	0.00	0.39	2.31	1.34	0.085	0.068
B4	0.00	0.15	0.70	0.61	0.078	0.063
B10	0.00	0.08	5.72	6.15	0.062	0.050
B12	0.00	0.00	0.99	3.42	0.042	0.033
B11	0.00	0.00	0.45	2.16	0.038	0.030
B9	0.00	0.37	0.92	2.46	0.060	0.048
B8	0.00	0.00	0.00	0.90	0.025	0.020
B13	0.00	0.00	0.85	4.22	0.038	0.030
B3	0.00	1.01	0.47	1.26	0.103	0.082
B2	0.00	0.01	0.28	1.65	0.037	0.030
B1	0.00	0.00	0.34	1.89	0.037	0.029
M3	0.00	0.38	0.49	4.89	0.043	0.034
M2	0.00	0.23	0.30	0.49	0.086	0.069
M1	0.00	0.14	1.35	2.00	0.061	0.049
LB15	0.00	0.00	0.04	0.31	0.033	0.026
LB10	0.00	0.00	0.00	0.52	0.025	0.020
LB20	0.00	0.00	0.00	0.82	0.025	0.020
LB30	0.00	0.00	0.00	0.50	0.025	0.020
LB35	0.00	0.00	0.00	0.81	0.025	0.020
LB40	0.00	0.00	0.03	1.61	0.027	0.021
PX1	0.00	4.92	0.72	1.83	0.148	0.118
LP10	0.00	0.47	0.41	0.37	0.115	0.092
LP20	0.03	0.24	0.43	0.12	0.126	0.101
LP30	0.00	0.04	2.02	0.00	0.102	0.081
LP31	0.00	0.00	0.30	0.80	0.045	0.036
LP34	0.00	0.00	1.24	0.00	0.100	0.080
LP32	0.00	0.00	0.34	0.05	0.090	0.072
LP33	0.00	0.00	0.25	0.06	0.085	0.068
LB50	0.00	0.00	0.51	0.99	0.051	0.040
LB55	0.00	0.00	0.19	0.07	0.079	0.064
LB60	0.00	0.00	0.55	0.00	0.100	0.080
LB70	0.00	0.00	0.23	0.00	0.100	0.080

Attachment 3- D. Bear Creek Subbasin Characteristics – Existing Conditions

Basin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss Rate	Constant Loss Rate	Impervious %	Associated Pump Station	Pump Station Capacity
		n	L			Lc	S	Lg						
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]		[cfs]
B7	30.24	0.2	14.49	950	131	6.57	56.52	12.59	FH	0.5	0.056	0	--	--
B6	11.73	0.2	8.50	466	114	4.30	41.39	9.29	FH	0.5	0.053	0	--	--
B5	4.04	0.18	5.05	227	97	2.21	25.72	5.83	VU	1.5	0.068	2	--	--
B4	1.53	0.15	1.96	96	93	1.11	1.53	4.47	VU	1.5	0.063	5	--	--
B10	12.01	0.18	5.37	302	111	2.61	35.59	5.97	FH	0.5	0.050	2	--	--
B12	4.41	0.18	5.32	165	80	2.85	15.99	7.17	FH	0.5	0.033	2	--	--
B11	2.60	0.18	2.98	150	88	1.64	20.84	4.43	FH	0.5	0.030	2	--	--
B9	3.88	0.18	5.44	111	71	3.17	7.35	8.73	VU	1.5	0.048	2	--	--
B8	0.95	0.18	1.68	80	75	0.68	2.98	3.70	VU	1.5	0.020	2	--	--
B13	5.28	0.15	5.87	140	71	2.59	11.76	6.34	VU	1.5	0.030	5	--	--
B3	2.84	0.18	3.79	93	69	1.78	6.33	6.29	VU	1.5	0.082	2	--	--
B2	2.06	0.18	2.81	70	55	1.28	5.35	5.11	VU	1.5	0.030	2	--	--
B1	2.30	0.18	1.86	55	45	0.87	5.37	3.78	VU	1.5	0.029	2	--	--
M3	5.76	0.18	4.49	166	95	2.64	15.81	6.55	FH	0.5	0.034	2	--	--
M2	1.02	0.18	2.18	92	75	1.24	7.80	4.26	VU	1.5	0.069	2	--	--
M1	3.54	0.15	2.76	75	55	1.68	7.25	4.42	VU	1.5	0.049	5	--	--
LB15	0.35	0.18	1.02	43	37	0.48	5.90	2.35	VU	1.5	0.026	2	--	--
LB10	0.54	0.18	1.04	45	37	0.57	7.68	2.40	VU	1.5	0.020	2	--	--
LB20	0.83	0.15	1.69	40	34	0.89	3.54	3.31	VU	1.5	0.020	5	--	--
LB30	0.50	0.18	1.38	35	25	0.68	7.25	2.89	VU	1.5	0.020	2	--	--
LB35	0.85	0.15	1.53	34	16	0.68	11.73	2.30	VU	1.5	0.020	2	--	--
LB40	1.69	0.18	1.87	29	13	0.57	8.57	2.94	VU	1.5	0.021	2	--	--
PX1	7.46	0.18	5.75	79	45	3.07	5.91	9.18	VU	1.5	0.118	2	--	--
LP10	1.25	0.15	1.27	45	40	0.69	3.92	2.65	VU	1.5	0.092	5	--	--
LP20	0.82	0.18	0.98	40	38	0.52	2.04	2.92	VU	1.5	0.101	2	--	--
LP30	2.09	0.18	2.58	38	24	1.29	5.43	4.95	VU	1.5	0.081	2	--	--
LP31	1.10	0.18	3.17	40	25	1.56	4.74	5.90	VU	1.5	0.036	2	--	--
LP32	0.40	0.18	0.80	21	20	0.37	1.24	2.61	VU	1.5	0.072	2	--	--
LP33	0.32	0.015	0.80	22	20	0.47	2.49	0.21	VD	0.2	0.068	60	Pixley PS	90.8
LB50	1.54	0.015	1.87	20	10	1.02	5.34	0.34	VD	0.2	0.040	60	Thornton PS	431
LB55	0.28	0.015	1.10	12	10	0.30	1.83	0.21	VD	0.2	0.064	60		
LB60	0.57	0.015	1.15	12	5	0.61	6.11	0.22	VD	0.2	0.080	60	I-5 PS	46.8
LB70	0.26	0.015	0.77	7	1	0.42	7.75	0.16	VD	0.2	0.080	60		

Notes: VU = Valley Undeveloped; VD = Valley Developed; FH = Foothill

Attachment 3- E. Bear Creek Subbasin Characteristics – Future Conditions

Basin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss Rate	Constant Loss Rate	Impervious %	Associated Pump Station	Pump Station Capacity
		n	L			Lc	S	Lg						
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]		[cfs]
B7	30.24	0.2	14.49	950.00	131.00	6.57	56.52	12.59	FH	0.5	0.056	0	--	--
B6	11.73	0.2	8.50	466.00	114.00	4.30	41.39	9.29	FH	0.5	0.053	0	--	--
B5	4.04	0.18	5.05	227.00	97.00	2.21	25.72	5.83	VU	1.5	0.068	2	--	--
B4	1.53	0.15	1.96	96.00	93.00	1.11	1.53	4.47	VU	1.5	0.063	5	--	--
B10	12.01	0.18	5.37	302.00	111.00	2.61	35.59	5.97	FH	0.5	0.050	2	--	--
B12	4.41	0.18	5.32	165.00	80.00	2.85	15.99	7.17	FH	0.5	0.033	2	--	--
B11	2.60	0.18	2.98	150.00	88.00	1.64	20.84	4.43	FH	0.5	0.030	2	--	--
B9	3.88	0.18	5.44	111.00	71.00	3.17	7.35	8.73	VU	1.5	0.048	2	--	--
B8	0.95	0.18	1.68	80.00	75.00	0.68	2.98	3.70	VU	1.5	0.020	2	--	--
B13	5.28	0.15	5.87	140.00	71.00	2.59	11.76	6.34	VU	1.5	0.030	5	--	--
B3	2.84	0.18	3.79	93.00	69.00	1.78	6.33	6.29	VU	1.5	0.082	2	--	--
B2	2.06	0.18	2.81	70.00	55.00	1.28	5.35	5.11	VU	1.5	0.030	2	--	--
B1	2.30	0.18	1.86	55.00	45.00	0.87	5.37	3.78	VU	1.5	0.029	2	--	--
M3	5.76	0.18	4.49	166.00	95.00	2.64	15.81	6.55	FH	0.5	0.034	2	--	--
M2	1.02	0.18	2.18	92.00	75.00	1.24	7.80	4.26	VU	1.5	0.069	2	--	--
M1	3.54	0.15	2.76	75.00	55.00	1.68	7.25	4.42	VU	1.5	0.049	5	--	--
LB15	0.35	0.015	1.02	43.00	37.00	0.48	5.90	0.20	VD	0.2	0.026	60	PLB15	83
LB10	0.54	0.015	1.04	45.00	37.00	0.57	7.68	0.20	VD	0.2	0.020	60	PLB10	128
LB20	0.83	0.015	1.69	40.00	34.00	0.89	3.54	0.33	VD	0.2	0.020	60	PLB20	197
LB30	0.50	0.015	1.38	35.00	25.00	0.68	7.25	0.24	VD	0.2	0.020	60	PLB30	118
LB35	0.85	0.015	1.53	34.00	16.00	0.68	11.73	0.23	VD	0.2	0.020	60	PLB35	201
LB40	1.69	0.015	1.87	29.00	13.00	0.57	8.57	0.25	VD	0.2	0.021	60	PLB40	400
PX1	7.46	0.115	5.75	79.00	45.00	3.07	5.91	5.87	VU	1.5	0.118	2	--	--
LP10	1.25	0.15	1.27	45.00	40.00	0.69	3.92	2.65	VU	1.5	0.092	5	--	--
LP20	0.82	0.18	0.98	40.00	38.00	0.52	2.04	2.92	VU	1.5	0.101	2	--	--
LP30	2.09	0.015	2.58	38.00	24.00	1.29	5.43	0.41	VD	0.2	0.081	60	PLP30	495
LP31	1.10	0.015	3.17	40.00	25.00	1.56	4.74	0.49	VD	0.2	0.036	60	PLP31	260
LP34	1.25	0.015	1.30	22.00	21.00	0.50	0.77	0.32	VD	0.2	0.080	60	PLP34	296
LP32	0.40	0.015	0.80	21.00	20.00	0.37	1.24	0.22	VD	0.2	0.072	60	PLP32	95
LP33	0.32	0.015	0.80	22.00	20.00	0.47	2.49	0.21	VD	0.2	0.068	60	Pixley PS	90.8
LB50	1.54	0.015	1.87	20.00	10.00	1.02	5.34	0.34	VD	0.2	0.040	60	Thornton PS	431
LB55	0.28	0.015	1.10	12.00	10.00	0.30	1.83	0.21	VD	0.2	0.064	60		
LB60	0.57	0.015	1.15	12.00	5.00	0.61	6.11	0.22	VD	0.2	0.080	60	I-5 PS	46.8
LB70	0.26	0.015	0.77	7.00	1.00	0.42	7.75	0.16	VD	0.2	0.080	60		

Notes: VU = Valley Undeveloped; VD = Valley Developed; FH = Foothill

Attachment 3- F. Bear Creek Depth-Duration-Frequency Tables

BEAR CREEK WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: SFS

Calculated Average Point Rainfall Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.134	0.176	0.209	0.254	0.288	0.322	0.358	0.405
10 min	0.193	0.252	0.300	0.364	0.413	0.462	0.513	0.581
15 min	0.233	0.305	0.363	0.440	0.499	0.559	0.620	0.702
30 min	0.320	0.419	0.498	0.604	0.685	0.767	0.851	0.964
60 min	0.443	0.579	0.689	0.836	0.948	1.062	1.178	1.335
3 hour	0.722	0.891	1.032	1.227	1.381	1.542	1.713	1.953
6 hour	0.983	1.193	1.369	1.614	1.809	2.012	2.229	2.536
12 hour	1.307	1.595	1.833	2.161	2.417	2.680	2.956	3.336
24 hour	1.793	2.218	2.564	3.032	3.392	3.756	4.132	4.641
48 hour	2.280	2.818	3.249	3.823	4.257	4.692	5.134	5.724
72 hour	2.623	3.236	3.725	4.372	4.857	5.341	5.829	6.478
96 hour	2.887	3.561	4.094	4.797	5.322	5.843	6.366	7.058

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
10 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
15 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
30 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
60 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
3 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
6 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
12 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
24 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
48 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
72 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
96 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.120	0.157	0.187	0.227	0.257	0.288	0.320	0.362
10 min	0.173	0.225	0.268	0.325	0.369	0.413	0.459	0.519
15 min	0.208	0.273	0.325	0.393	0.446	0.500	0.554	0.628
30 min	0.286	0.375	0.445	0.540	0.612	0.686	0.761	0.862
60 min	0.396	0.518	0.616	0.747	0.848	0.949	1.053	1.193
3 hour	0.645	0.797	0.923	1.097	1.235	1.379	1.531	1.746
6 hour	0.879	1.067	1.224	1.443	1.617	1.799	1.993	2.267
12 hour	1.168	1.426	1.639	1.932	2.161	2.396	2.643	2.982
24 hour	1.603	1.983	2.292	2.711	3.032	3.358	3.694	4.149
48 hour	2.038	2.519	2.905	3.418	3.806	4.195	4.590	5.117
72 hour	2.345	2.893	3.330	3.909	4.342	4.775	5.211	5.791
96 hour	2.581	3.184	3.660	4.289	4.758	5.224	5.691	6.310

BEAR CREEK WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: SFS

Calculated Average Point Rainfall Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.134	0.176	0.209	0.254	0.288	0.322	0.358	0.405
10 min	0.193	0.252	0.300	0.364	0.413	0.462	0.513	0.581
15 min	0.233	0.305	0.363	0.440	0.499	0.559	0.620	0.702
30 min	0.320	0.419	0.498	0.604	0.685	0.767	0.851	0.964
60 min	0.443	0.579	0.689	0.836	0.948	1.062	1.178	1.335
3 hour	0.722	0.891	1.032	1.227	1.381	1.542	1.713	1.953
6 hour	0.983	1.193	1.369	1.614	1.809	2.012	2.229	2.536
12 hour	1.307	1.595	1.833	2.161	2.417	2.680	2.956	3.336
24 hour	1.793	2.218	2.564	3.032	3.392	3.756	4.132	4.641
48 hour	2.280	2.818	3.249	3.823	4.257	4.692	5.134	5.724
72 hour	2.623	3.236	3.725	4.372	4.857	5.341	5.829	6.478
96 hour	2.887	3.561	4.094	4.797	5.322	5.843	6.366	7.058

Upper Watershed Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.733	0.738	0.738	0.736	0.736	0.736	0.736	0.735
10 min	0.735	0.736	0.738	0.736	0.737	0.735	0.734	0.734
15 min	0.735	0.736	0.736	0.737	0.737	0.736	0.735	0.733
30 min	0.734	0.737	0.737	0.736	0.736	0.736	0.734	0.732
60 min	0.735	0.738	0.738	0.738	0.738	0.736	0.736	0.735
3 hour	0.735	0.735	0.735	0.735	0.734	0.733	0.733	0.732
6 hour	0.735	0.734	0.734	0.733	0.733	0.732	0.732	0.732
12 hour	0.735	0.734	0.734	0.733	0.733	0.733	0.733	0.733
24 hour	0.735	0.735	0.735	0.735	0.735	0.735	0.735	0.735
48 hour	0.736	0.735	0.735	0.735	0.735	0.735	0.736	0.736
72 hour	0.736	0.736	0.735	0.735	0.735	0.735	0.736	0.736
96 hour	0.736	0.735	0.735	0.735	0.735	0.735	0.735	0.736

Upper Watershed Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.098	0.130	0.154	0.187	0.212	0.237	0.263	0.298
10 min	0.142	0.185	0.221	0.268	0.304	0.340	0.377	0.426
15 min	0.171	0.224	0.267	0.324	0.368	0.411	0.456	0.515
30 min	0.235	0.309	0.367	0.445	0.504	0.565	0.625	0.706
60 min	0.326	0.427	0.508	0.617	0.700	0.782	0.867	0.981
3 hour	0.531	0.655	0.759	0.902	1.014	1.130	1.256	1.430
6 hour	0.723	0.876	1.005	1.183	1.326	1.473	1.632	1.856
12 hour	0.961	1.171	1.345	1.584	1.772	1.964	2.167	2.445
24 hour	1.318	1.630	1.885	2.229	2.493	2.761	3.037	3.411
48 hour	1.678	2.071	2.388	2.810	3.129	3.449	3.779	4.213
72 hour	1.931	2.382	2.738	3.213	3.570	3.926	4.290	4.768
96 hour	2.125	2.617	3.009	3.526	3.912	4.295	4.679	5.195

BEAR CREEK WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: RBR

Calculated Average Point Rainfall Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.140	0.179	0.212	0.259	0.295	0.333	0.374	0.431
10 min	0.201	0.257	0.305	0.371	0.423	0.478	0.536	0.618
15 min	0.243	0.311	0.368	0.448	0.512	0.578	0.648	0.747
30 min	0.337	0.433	0.512	0.623	0.711	0.803	0.901	1.039
60 min	0.456	0.585	0.692	0.842	0.962	1.086	1.218	1.404
3 hour	0.751	0.929	1.080	1.294	1.467	1.649	1.846	2.125
6 hour	1.029	1.260	1.454	1.728	1.948	2.178	2.424	2.773
12 hour	1.377	1.692	1.952	2.312	2.593	2.884	3.187	3.606
24 hour	1.882	2.331	2.695	3.188	3.565	3.947	4.338	4.867
48 hour	2.368	2.936	3.388	3.988	4.438	4.886	5.338	5.935
72 hour	2.718	3.372	3.888	4.565	5.068	5.564	6.061	6.712
96 hour	2.997	3.722	4.290	5.030	5.576	6.112	6.645	7.339

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.767	0.764	0.764	0.765	0.765	0.765	0.765	0.766
10 min	0.766	0.765	0.764	0.765	0.765	0.766	0.766	0.767
15 min	0.766	0.765	0.765	0.765	0.765	0.765	0.766	0.767
30 min	0.766	0.765	0.765	0.765	0.765	0.765	0.766	0.767
60 min	0.766	0.764	0.764	0.764	0.764	0.765	0.765	0.766
3 hour	0.766	0.766	0.766	0.766	0.767	0.767	0.767	0.768
6 hour	0.766	0.766	0.766	0.767	0.767	0.767	0.767	0.768
12 hour	0.766	0.766	0.767	0.767	0.767	0.767	0.767	0.767
24 hour	0.766	0.766	0.766	0.766	0.766	0.766	0.766	0.766
48 hour	0.765	0.766	0.766	0.766	0.766	0.766	0.766	0.765
72 hour	0.765	0.766	0.766	0.766	0.766	0.766	0.765	0.765
96 hour	0.765	0.766	0.766	0.766	0.766	0.766	0.766	0.765

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.107	0.137	0.162	0.198	0.226	0.255	0.286	0.330
10 min	0.154	0.197	0.233	0.284	0.324	0.366	0.411	0.474
15 min	0.186	0.238	0.282	0.343	0.392	0.442	0.496	0.573
30 min	0.258	0.331	0.392	0.477	0.544	0.614	0.690	0.797
60 min	0.349	0.447	0.529	0.643	0.735	0.831	0.932	1.075
3 hour	0.575	0.712	0.827	0.991	1.125	1.265	1.416	1.632
6 hour	0.788	0.965	1.114	1.325	1.494	1.671	1.859	2.130
12 hour	1.055	1.296	1.497	1.773	1.989	2.212	2.444	2.766
24 hour	1.442	1.786	2.064	2.442	2.731	3.023	3.323	3.728
48 hour	1.812	2.249	2.595	3.055	3.400	3.743	4.089	4.540
72 hour	2.079	2.583	2.978	3.497	3.882	4.262	4.637	5.135
96 hour	2.293	2.851	3.286	3.853	4.271	4.682	5.090	5.614

BEAR CREEK WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: RBR

Calculated Average Point Rainfall Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.140	0.179	0.212	0.259	0.295	0.333	0.374	0.431
10 min	0.201	0.257	0.305	0.371	0.423	0.478	0.536	0.618
15 min	0.243	0.311	0.368	0.448	0.512	0.578	0.648	0.747
30 min	0.337	0.433	0.512	0.623	0.711	0.803	0.901	1.039
60 min	0.456	0.585	0.692	0.842	0.962	1.086	1.218	1.404
3 hour	0.751	0.929	1.080	1.294	1.467	1.649	1.846	2.125
6 hour	1.029	1.260	1.454	1.728	1.948	2.178	2.424	2.773
12 hour	1.377	1.692	1.952	2.312	2.593	2.884	3.187	3.606
24 hour	1.882	2.331	2.695	3.188	3.565	3.947	4.338	4.867
48 hour	2.368	2.936	3.388	3.988	4.438	4.886	5.338	5.935
72 hour	2.718	3.372	3.888	4.565	5.068	5.564	6.061	6.712
96 hour	2.997	3.722	4.290	5.030	5.576	6.112	6.645	7.339

Upper Watershed Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
10 min	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
15 min	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
30 min	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
60 min	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
3 hour	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
6 hour	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
12 hour	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
24 hour	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
48 hour	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
72 hour	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848
96 hour	0.848	0.848	0.848	0.848	0.848	0.848	0.848	0.848

Upper Watershed Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.119	0.152	0.180	0.220	0.250	0.282	0.317	0.365
10 min	0.170	0.218	0.259	0.315	0.359	0.405	0.455	0.524
15 min	0.206	0.264	0.312	0.380	0.434	0.490	0.550	0.633
30 min	0.286	0.367	0.434	0.528	0.603	0.681	0.764	0.881
60 min	0.387	0.496	0.587	0.714	0.816	0.921	1.033	1.191
3 hour	0.637	0.788	0.916	1.097	1.244	1.398	1.565	1.802
6 hour	0.873	1.068	1.233	1.465	1.652	1.847	2.056	2.352
12 hour	1.168	1.435	1.655	1.961	2.199	2.446	2.703	3.058
24 hour	1.596	1.977	2.285	2.703	3.023	3.347	3.679	4.127
48 hour	2.008	2.490	2.873	3.382	3.763	4.143	4.527	5.033
72 hour	2.305	2.859	3.297	3.871	4.298	4.718	5.140	5.692
96 hour	2.541	3.156	3.638	4.265	4.728	5.183	5.635	6.223

Attachment 3- G. ITR Comment Forms for Bear Creek HEC-HMS Modeling

Lower San Joaquin River Feasibility Study

HYDROLOGIC MODELING REVIEW – BEAR CREEK WATERSHED

Task Order 6 – LSJRFS Work-In-Kind Hydrology

Reviewer:	Domenichelli & Associates
Review Date:	8-23-10
PBI Response Date:	9-24-10
Domenichelli Backcheck:	10-8-10

Backcheck Comments:

1. All previous comments were addressed adequately. No back check comments on previous comments.

2. New calibration information (Section 3.5): In the text it is stated that the new calibration gage data in Figure 8 corresponds to HMS model element MSRTN. The Figure shows a peak 100-yr event flow of approx 4,100cfs, however the model provided shows a peak flow of 6,300cfs at element MSRTN. Why is there a different result in the model?

   PBI Response: Figure 8 displays the model calibration results at MSRTN. The calibration utilized an observed rainfall event taken from historical gage records. The model provided to D&A includes the 100-year rainfall event taken from the 1998 SJAFCA HEC-1 model. Results shown in Figure 8 are expected to differ from the reviewed model due to differing rainfall inputs.

Lower San Joaquin River Feasibility Study

HYDROLOGIC MODELING REVIEW – BEAR CREEK WATERSHED

Task Order 6 – LSJRFS Work-In-Kind Hydrology

Reviewer:	Domenichelli & Associates
Review Date:	8-23-10
PBI Response Date:	9-24-10

Note to Reviewer: After the original Draft TM was sent to D&A for ITR, PBI was able to obtain more detailed calibration data. Section 3.5 is now updated to describe the latest calibration methodology. Although new comments are not usually part of the backcheck process, comments on Section 3.5 are welcome if needed.

Memorandum Comments:

1. *Section 3.2 Model Development - This would be a good place early in the TM to describe the "Existing Conditions" and "Future Without Project" model assumptions and parameters.*

PBI Response: Section 3.2 now includes a mention of the 'Existing' and 'Future Without Project' model runs. However, this section was not intended to include significant details on the assumptions and parameters of the model runs and was only meant to provide an outline of the TM. Instead, Sections 3.6.1 and 3.6.2 include the relevant details and were referenced.

2. *Section 3.2 , Page 2, Item 3.- Provide Reference…*

PBI Response: Agreed. See Section 3.2 , Page 2, Item 3.

3. *Section 3.2.1, Paragraph 3 - Remove "was used" after "method".*

PBI Response: Agreed. See Section 3.2.1 , Paragraph 3.

4. *Section 3.3, Design Storms - Provide reasoning for 3-day storm and reason for (use of) so many (8) design frequency storms.*

PBI Response: Additional reasoning was added to Section 3.3 which now reads:
"…A 72-hour storm was selected to stress the basin from both a peak flow and volume standpoint.…"
-and-

"The selection of eight design storms provides a wide range of scenarios that can be used for planning purposes."

5. *Table 2, Consider providing a column for routing reach description (ie..Basin B7 to B6).*

PBI Response: Agreed. See Section 3.4.5, Page 12, Table 2.

6. *Section 3.4.6, Loss Rates- The loss rates will usually vary with different soils type, natural cover, etc. Only one initial and constant rate combination is used. Is the entire Bear Creek watershed Type D soil? May want to include a soils map somewhere for confirmation.*

PBI Response: The method for assigning constant loss rates has now been modified. GIS soils layers were obtained from the NRCS. The percentage of hydrologic soil groups (A, B, C, or D) contained within each subbasin was determined through GIS calculations. Soil groups were each assigned a loss rate based on published studies and a weighted loss rate was calculated for each subbasin.

7. *Section 3.4.7 Impervious Percentages- May consider more intermediate values of impervious percentage to meet varied land uses (especially under existing, non built out conditions). Some adjacent upper sheds change between 2% to 10% with only small changes in current development.*

PBI Response: After further discussions with Domenichelli & Associates, impervious percentages for the 'Agricultural with Rural Development' land use classification were changed from 10% to 5% to ensure that no subbasins east of the CCTR are assigned impervious percentages greater than 5%.

8. *Section 3.6.2 Future Without Project Conditions- Is 2070 a typo error or does some document estimate the level of development for that time (seems like an odd number.)*

PBI Response: 2070 is the agreed upon Future-Without-Project date. The 2035 general plan gives the best possible estimates available for land use conditions given that a 2070 general plan does not exist.

9. *Attachment A – There is no reference to this table in the text. Will it be referenced under 3.7 Model Results after the models are updated with new rainfall data? Will a table for existing conditions be provided at that time also?*

PBI Response: Attachment A is referenced in Section 3.2.2, Page 4, Paragraph 4. It is meant to compare the SJAFCA HEC-1 model's results to the results produced by an "interim" HEC-HMS model which was produced after directly converting from the HEC-1 model. Note that this interim HMS model is not the most updated model which includes the initial/constant loss rate method, etc. Instead, these interim HMS results reflect a model that uses the same methods (Curve Numbers, etc) as the SJAFCA HEC-1 model and are meant to show that the direct conversion went smoothly.

Once the models are updated with the new rainfall data produced from the NOAA study, both ‘Existing’ and ‘Future Without Project’ production runs will be performed and results tables with be provided in a subsequent TM.

10. *Attachment A –label Table as “Future Without Project” for clarity. May also consider adding a column to describe the model elements.*

PBI Response: See PBI response to Item 9.

Once ‘Existing’ and ‘Future-Without-Project’ production runs are performed, complete results tables will be provided as Attachments. In addition, summary tables of model results will be provided in Section 3.7 of the final report. These tables will include peak flows produced at key locations in the watershed along with a description of model element locations.

11. *Attachment A – How do the PBI, HMS sub-basin flows shown in the table match precisely with the SJAFCA sub-basin results, even though a different loss rate method was used and more precise topo is used for lag time calculations? Would have expected some deviation in these comparisons. (See following comment 12)*

PBI Response: See PBI response to Item 9.

12. *Latest Model Results - Attachment A results in the table do not match the results provided in the latest HMS model.*

PBI Response: See PBI response to Item 9.

13. *Latest Model Results- Latest HMS results are significantly lower than the SJAFCA results at the downstream end of the system (approximately 18% lower), even though the flow at the Lockford gage was only 2% lower and one more sub-basin (LP34) is added to the new model at the downstream end. What is/are the reason(s) for this difference. The difference should be explained in the text.*

PBI Response: The HMS model was calibrated to give the same results at the Lockeford gage. However this location only includes 3 subbasins in its drainage area. The remaining 29 subbasins are expected to give differing results from the HEC-1 model for the following reasons:

1. The loss method was changed which fundamentally changes the calculations for infiltration.
2. The lag times were re-calculated for the PBI Model and, in many cases, are significantly different from those entered in the SJAFCA HEC-1 model. Both the magnitude and timing of peak flows are affected by the change in lag times. Therefore the peak flow contributions from the subbasins are expected to arrive at the model outlet with different timing and magnitudes than what occurred in the SJAFCA HEC-1 model.
3. PBI's 'Future-Without-Project' model has 5 additional pumps compared to the SJAFCA HEC-1 model. These pumps are set to discharge at 0.37 cfs/acre of tributary area (roughly a 1/10-AEP flow) and regulate flows for 5 additional subbasins which would otherwise contribute much higher peak flows.

14. *Figure 2 – Text box reads "LP24" should read "LP34"*

PBI Response: Agreed. See Figure 2.

Attachment 3- H. SPK Comment Forms for Bear Creek HEC-HMS Modeling

Corps of Engineers, Hydrology Section

Review of Bear Creek HEC-1 to HEC-HMS model conversion and preliminary report.
12 November 2010 with Responses 03 December 2010, SFH

Steven F. Holmstrom, P.E.

The Technical Memorandum for the Lower San Joaquin River Feasibility Study Bear Creek HEC-HMS modeling DRAFT hydrology report has been reviewed and the following comments are provided.

1. Section 3.2.1, SJAFCA HEC-1 model, states that the 1/100 AEP rainfall event matched the 1/100 AEP peak flow from the Bear Creek at Lockeford stream gage. Information on the period of record and statistics (mean, SD, skew), should be shown in the report to help quantify the uncertainty in the period of record. If additional data is available, the frequency curve should be updated to reduce the uncertainty in the estimate. The frequency curve used should be included in the report.

   PBI Response: The Lockeford stream gage was used in the calibration of the 1998 SJAFCA HEC-1 model. The frequency curve and statistics for this gage are now provided in Attachment B.

   SPK backcheck: OK

2. In section 3.3 Design Storms, the fourth paragraph states that two storm centerings will be analyzed. A third storm centering will be required to compute the flow above the New Hogan Dam for the 8 design storm frequencies.

   PBI Response: It is PBI's understanding that the third storm centering above New Hogan Dam will only be analyzed for the Calaveras River watershed. No changes were made to this section.

   SPK backcheck: OK

3. Section 3.4.1, subbasins, states that DWR LiDAR[2] data were used to confirm and revise subbasin boundaries and drainage areas. In addition, the study states that other HMS parameters were adjusted to calibrate the runoff at the gage. The study must provide a table showing the adjusted drainage area, and the differences in input parameters between the HEC-1 and re-calibrated HMS models.

   PBI Response: Parameters from the 1998 SJAFCA HEC-1 model are now included as Attachment A.

SPK backcheck: Attachment A appears to have changed from a comparison of peak flows from HEC-1 to HMS to a tabulation of Watershed parameters from the 1998 report. What I am looking for are 3 additional columns in the table comparing peak flows, that show the drainage area used in the 1998 study and the REVISED drainage area (which accounts for a difference in peak flow) to be used in the current study. Please restore the table in attachment B from the first draft and add columns representing DA from 1998, DA from GeoHMS and percent difference.

PBI Response to Backcheck: Attachment A now includes a subbasin parameter comparison between the 1998 HEC-1 model and the 2010 PBI HEC-HMS Model. When I included the HEC-1 vs. HEC-HMS peak flow results in the original Attachment A that you referenced, it seemed to cause quite a bit of confusion both in the ITR Review and SPK Review.

To clarify, the HEC-HMS model that produced the results listed in that Attachment used the same subbasin areas, same lag times, same curve number method, etc. as the 1998 HEC-1 model. There were virtually no differences between the peak flows listed in the two columns. This table was intended to show that the HEC-1 data cards were uploaded correctly into HEC-HMS.

In an attempt to eliminate any further confusion, I took the peak flow comparison table out of the report because most reviewers mistakenly thought that the peak flow results listed in the HEC-HMS column were from the 2010 PBI HEC-HMS Model.

Further Comparisons of HEC-1 and HEC-HMS results can be included once the final NOAA14 rainfall data are incorporated into the 2010 PBI HEC-HMS Model.

4. On figure 2, the Bear Creek at Lockeford stream gage location should be identified, or noted to be co-located with the ALERT gage. In addition, the location of flows diverted from Mosher Creek should be identified as input to Bear Creek.

   PBI Response: The Bear Creek at Lockeford stream gage is now labeled in Figure 2. Note that the Lockeford gage and the ALERT gage are not the same gage. The Bear Creek ALERT gage on is located approximately 7 miles downstream from the Lockeford gage.

   SPK backcheck: OK

5. In section 3.4.3, Diversions, the study should discuss the possibility of the diversions being overwhelmed during very high flows that is greater than the 1% flood.

   PBI Response: Section 3.4.3 has been updated.

   SPK backcheck: OK

6. On figure 6, Bear Creek subbasin flow paths, the location where Mosher Slough subbasin 1105 (M1, M2, M3) is diverted into Bear Creek must be shown. The subbasin boundary should also be shown in the Bear Creek figures.

   PBI Response: The Mosher Slough diversion point is now labeled in Figures 2 and 6. Note that Bear Creek's subbasins M1, M2, and M3 and Mosher Slough's subbasin 1105 do not cover the same drainage area; they are adjacent to one another. Mosher Slough subbasin 1105 was therefore not included in Figure 6.

   SPK backcheck: OK

7. The subbasin lag times defined in section 3.4.4, unit hydrograph S-graph and lag times, and listed in attachment C, subbasin characteristics, use a basin 'n in the calculations that appears to be high. Figure E-2 in the San Joaquin Hydrology Manual, notes that a basin 'n' of 0.20 is appropriate where "the groundcover consists of cultivated crops", and where the "surface characteristics are such that channelization does not occur". It appears that the choice of high basin 'n' values may result in very low (0.02 "/hour) loss rates. The study must review the relationship between basin 'n' values and loss rates through sensitivity analysis to derive more rational values for each parameter.

   PBI Response: As noted in Figure E-2 in San Joaquin's Hydrology Manual, a basin 'n' of 0.2 is appropriate for areas with cultivated crops where channelization does not occur. PBI assigned a basin 'n' value of 0.2 for all agricultural lands that have relatively flat slopes. When looking at the descriptions in the Hydrology Manual, 0.2 would appear to be the most appropriate basin 'n' value for flat, agricultural land. This land has "cultivated crops" and "channelization does not occur" due to its flat surface. If, on the other hand, the land had steeper slopes or was not cultivated, a basin 'n' value of 0.2 would be too high.

   Note that the assignment of basin 'n' values and loss rates are independent of each other. Basin 'n' values are assigned based on the land use type, etc. and are used in calculations for basin lag times (see Section 3.4.4). Loss rates are assigned based on the soil makeup within each subbasin (see Section 3.4.6). The low loss rates are a result of the abundance of Type D soils seen throughout the watershed.

   SPK backcheck: OK

8. The frequency of the Jan-Feb 1998 event noted in section 3.5, model calibration, must be shown. This information may be included on the Bear Creek at Lockeford frequency curve mentioned in item 1 above.

   PBI Response: A mention of the frequency of the calibration event is now included in Section 3.5. Note that the calibration location is approximately 7 miles downstream from the Lockeford gage. The Lockeford frequency curve was adjusted based on the proportional relationship between its drainage area and the drainage area at the calibration location. This adjusted curve was then used to estimate that the calibration event is approximately a 1/10 AEP event.

SPK backcheck: Clarify the duration of "effective rainfall" that produced the peak flow, in section 3.5. A 12-day duration sounds high relative to the 3-day (72-hour) duration selected for the current study.
PBI Response to Backcheck: Section 3.5 has been updated.

9. In section 3.6.1, Model Simulations, Existing Conditions, the statement is made that "any subbasin flows exceeding pump station capacities would result in temporary ponding within the subbasin." The study must be clear that this condition is not related to exterior stages in the receiving stream. Or a coincidence analysis must be performed to relate interior and exterior stages.

   PBI Response: Section 3.6.1 has been updated.

   SPK backcheck: OK

10. Subbasin 1105 (M1, M2, M3) from Mosher Slough must be added to the table in attachment A which compares the HEC1 and HMS results. This (1105) or these (M1, M2, M3) subbasin(s) should also be included in the final results tables.

    PBI Response: As noted above in Response #6, Bear Creek subbasins M1, M2, and M3 do not cover the same area as Mosher Slough subbasin 1105; these subbasins are adjacent to one another. Mosher Slough subbasin 1105 was therefore not included in any of the final results table because it is not part of the Bear Creek model.

    SPK backcheck: OK

11. The HMS model transmitted with the report does not appear to match either the results from the 1998 study or the calibration done for the current study. The report must clarify the purpose and the state of the input parameters contained in the model supplied.

    PBI Response: The HMS model transmitted with the report is not expected to match the results from the 1998 study or from the calibration run.

    The results table originally included in Attachment A was meant to show that the conversion from the HEC-1 model to HEC-HMS went smoothly. The HEC-HMS results listed in that table were for a model that still used the old methodology from the 1998 HEC-1 study (curve numbers, etc). This table has now been excluded from PBI's report so as to avoid any confusion.

The HMS model transmitted will also not produce the same results as the calibration shown in Figure 8. The calibration was run using observed rainfall data from a historical storm whereas the transmitted model is coded with the 100-year design storms that were used in the 1998 HEC-1 model.

The input parameters for the transmitted model are listed in Attachment D & Attachment E. Once the NOAA14 design storms are determined, they will be coded into the PBI Model.

SPK backcheck: OK

Attachment 4- A. Mosher Slough Watershed Subbasin Parameters Used in the 1998 SJAFCA HEC-1 Model

MOSHER SLOUGH WATERSHED PARAMETERS

Parameter	Description	Basin													
		1105	1104	1103C	1103B	1103A	CHER	CAY	ELD	THOR	LSAC	ROYAL	DON
DA,mi.^2	Drainage Area	1.21	2.69	0.67	0.145	0.034	1.69	1.11	0.73	0.2	0.34	0.7	0.7
XL, mi.	Watercourse Length	1.9	4.5	1.14	0.95	0.49	3.78	1.8	1.27	0.47	0.47	0.63	0.63
XLCA, mi.	Length from Centroid	0.95	1.8	0.61	0.66	0.25	1.89	0.28	0.06	0.19	0.19	0.19	0.09
S, ft./mi.	Subarea Slope	3.96	12.1	5.15	5.81	7	5.28	4.22	3.7	3.17	3.17	6.86	7.92
BN	Basin "n" Value	0.08	0.16	0.14	0.14	0.14	0.025	0.025	0.025	0.025	0.025	0.025	0.025
ARF, mi^2	Storm Area Reduction Factor	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
MAP, in.	Mean Annual Precipitation	16.5	15.5	14.7	14.7	14.7	15.5	15.5	14.7	14.0	14.0	14.0	14.0
Peak, cfs	Peak Flow	322.1	264.7	175.5	42.9	17.9	952.2	1508.4	1484.9	374.7	636.9	1359.8	1663.2
Lag, hrs.	Calculated Basin Lag Time	1.85	5.29	2.14	2.01	1.05	0.92	0.35	0.18	0.19	0.19	0.19	0.14
SCS Num.	NCRS Curve Number	81	81	86	86	86	86	86	86	86	86	86	86
S Curve		VU	VU	VD	VD	VD	VD	VD	VD	VD	VD	VD	VD

Parameter	Description	Basin		
		YAR	BAIN	KELLY
DA,mi.^2	Drainage Area	0.25	0.12	0.82
XL, mi.	Watercourse Length	0.57	0.34	1.14
XLCA, mi.	Length from Centroid	0.15	0.19	0.57
S, ft./mi.	Subarea Slope	3.17	5.28	5.28
BN	Basin "n" Value	0.025	0.025	0.025
ARF, mi^2	Storm Area Reduction Factor	N/A	N/A	N/A
MAP, in.	Mean Annual Precipitation	14.0	14.0	14.0
Peak, cfs	Peak Flow	477.6	269.2	1085.0
Lag, hrs.	Calculated Basin Lag Time	0.19	0.15	0.37
SCS Num.	NCRS Curve Number	86	86	86
S Curve		VD	VD	VD

S Curve Designations:

FH Foothill S-Curve
VU Valley Undeveloped
VD Valley Developed

Source: HDR, Final Technical Memorandum #1, Hydrologic Report prepared for San Joaquin Area Flood Control Agency, January 1998.

Attachment 4- B. Mosher Slough Subbasin Soil Groups and Loss Rates

Soil Group:	A (0.35 in/hr)	B (0.2 in/hr)	C (0.1 in/hr)	D (0.025 in/hr)	Composite Loss Rate	Adjusted Loss Rate (Adjustment Factor = 0.80)
Subbasin	[sq. mi.]	[sq. mi.]	[sq. mi.]	[sq. mi.]	[in/hr]	[in/hr]
1104	0.00	0.01	0.43	2.82	0.035	0.028
1105	0.00	0.00	0.67	1.07	0.054	0.043
1103C	0.00	0.00	0.00	1.00	0.025	0.020
1103B	0.00	0.00	0.00	0.24	0.025	0.020
1103D	0.00	0.00	0.00	1.17	0.025	0.020
1103A	0.00	0.00	0.00	0.10	0.025	0.020
CHER	0.00	0.00	0.00	1.78	0.025	0.020
CAY	0.00	0.00	0.00	1.17	0.025	0.020
ELD	0.00	0.00	0.00	0.71	0.025	0.020
THOR	0.00	0.00	0.00	0.47	0.025	0.020
ROYAL	0.00	0.00	0.00	0.72	0.025	0.020
LSAC	0.00	0.00	0.00	0.35	0.025	0.020
DON	0.00	0.00	0.44	0.52	0.059	0.047
BAIN	0.00	0.00	0.14	0.00	0.100	0.080
KELLY	0.00	0.00	0.67	0.10	0.090	0.072
YAR	0.00	0.00	0.28	0.02	0.095	0.076
TCREEKS	0.00	0.00	0.17	0.00	0.100	0.080
ATLAS	0.00	0.00	0.47	0.00	0.100	0.080

Attachment 4- C. Mosher Slough Subbasin Characteristics – Existing Conditions

Basin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
1104	3.25	0.15	3.66	54	35	1.44	5.20	4.95	VU	1.5	0.028	10	85.3
1105	1.75	0.2	2.26	64	50	0.77	6.18	4.19	VU	1.5	0.043	2	
1103C	1.04	0.015	2.18	40	31	1.27	4.13	0.40	VD	1.5	0.020	60	
1103B	0.24	0.015	1.07	35	30	0.60	4.68	0.23	VD	1.5	0.020	60	
1103D	1.17	0.015	1.19	30	26	0.44	3.36	0.22	VD	1.5	0.020	60	
1103A	0.10	0.015	0.59	30	26	0.31	6.82	0.13	VD	1.5	0.020	60	
CHER	1.78	0.015	1.50	25	20	0.48	3.34	0.25	VD	1.5	0.020	60	199.5
CAY	1.17	0.015	2.01	21	20	0.63	0.50	0.45	VD	1.5	0.020	60	269.2
ELD	0.71	0.015	1.55	20	16	0.77	2.57	0.32	VD	1.5	0.020	60	188.5
THOR	0.47	0.015	0.86	11	10	0.47	1.16	0.25	VD	1.5	0.020	60	26.8
ROYAL	0.73	0.015	0.88	19	13	0.17	6.80	0.12	VD	1.5	0.020	60	204.5
LSAC	0.35	0.015	0.75	20	15	0.40	6.67	0.16	VD	1.5	0.020	60	19.0
DON	0.96	0.015	1.26	14	1	0.55	10.28	0.20	VD	1.5	0.047	60	77.7
BAIN	0.14	0.015	0.69	2	1	0.37	1.46	0.20	VD	1.5	0.080	60	43.5
KELLY	0.79	0.015	1.16	10	1	0.61	7.76	0.21	VD	1.5	0.072	60	152.6
YAR	0.30	0.015	1.05	5	1	0.60	3.83	0.23	VD	1.5	0.076	60	82.1
TCREEKS	0.17	0.015	0.73	1	0	0.23	1.37	0.17	VD	1.5	0.080	60	34.8
ATLAS	0.51	0.115	1.09	1	0	0.59	0.92	2.37	VU	1.5	0.080	2	--

Notes: VU = Valley Undeveloped; VD = Valley Developed

Attachment 4- D. Mosher Slough Subbasin Characteristics – Future Conditions

Basin	Area [Sq. Mi.]	Basin 'n' n	Watercourse Length L [miles]	Upstream Elevation [feet]	Downstream Elevation [feet]	Length from Centroid Lc [miles]	Watercourse Slope S [ft/mile]	Lag Time Lg [hrs]	S-Graph	Initial Loss [inches]	Constant Loss Rate [in/hour]	Impervious % [%]	Pump Station Capacity [cfs]
1104	3.25	0.15	3.66	54	35	1.44	5.20	4.95	VU	1.5	0.028	10	85.3
1105	1.75	0.20	2.26	64	50	0.77	6.18	4.19	VU	1.5	0.043	2	
1103C	1.04	0.015	2.18	40	31	1.27	4.13	0.40	VD	1.5	0.020	60	
1103B	0.24	0.015	1.07	35	30	0.60	4.68	0.23	VD	1.5	0.020	60	
1103D	1.17	0.015	1.19	30	26	0.44	3.36	0.22	VD	1.5	0.020	60	
1103A	0.10	0.015	0.59	30	26	0.31	6.82	0.13	VD	1.5	0.020	60	
CHER	1.78	0.015	1.50	25	20	0.48	3.34	0.25	VD	1.5	0.020	60	199.5
CAY	1.17	0.015	2.01	21	20	0.63	0.50	0.45	VD	1.5	0.020	60	269.2
ELD	0.71	0.015	1.55	20	16	0.77	2.57	0.32	VD	1.5	0.020	60	188.5
THOR	0.47	0.015	0.86	11	10	0.47	1.16	0.25	VD	1.5	0.020	60	26.8
ROYAL	0.73	0.015	0.88	19	13	0.17	6.80	0.12	VD	1.5	0.020	60	204.5
LSAC	0.35	0.015	0.75	20	15	0.40	6.67	0.16	VD	1.5	0.020	60	19.0
DON	0.96	0.015	1.26	14	1	0.55	10.28	0.20	VD	1.5	0.047	60	77.7
BAIN	0.14	0.015	0.69	2	1	0.37	1.46	0.20	VD	1.5	0.080	60	43.5
KELLY	0.79	0.015	1.16	10	1	0.61	7.76	0.21	VD	1.5	0.072	60	152.6
YAR	0.30	0.015	1.05	5	1	0.60	3.83	0.23	VD	1.5	0.076	60	82.1
TCREEKS	0.17	0.015	0.73	1	0	0.23	1.37	0.17	VD	1.5	0.080	60	34.8
ATLAS	0.51	0.015	1.09	1	0	0.59	0.92	0.31	VD	1.5	0.080	60	120.8

Notes: VU = Valley Undeveloped; VD = Valley Developed

Attachment 4- E. Mosher Slough Depth-Duration-Frequency Tables

MOSHER SLOUGH WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SFS

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.134	0.176	0.209	0.254	0.288	0.322	0.358	0.405
10 min	0.193	0.252	0.300	0.364	0.413	0.462	0.513	0.581
15 min	0.233	0.305	0.363	0.440	0.499	0.559	0.620	0.702
30 min	0.320	0.419	0.498	0.604	0.685	0.767	0.851	0.964
60 min	0.443	0.579	0.689	0.836	0.948	1.062	1.178	1.335
3 hour	0.722	0.891	1.032	1.227	1.381	1.542	1.713	1.953
6 hour	0.983	1.193	1.369	1.614	1.809	2.012	2.229	2.536
12 hour	1.307	1.595	1.833	2.161	2.417	2.680	2.956	3.336
24 hour	1.793	2.218	2.564	3.032	3.392	3.756	4.132	4.641
48 hour	2.280	2.818	3.249	3.823	4.257	4.692	5.134	5.724
72 hour	2.623	3.236	3.725	4.372	4.857	5.341	5.829	6.478
96 hour	2.887	3.561	4.094	4.797	5.322	5.843	6.366	7.058

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
10 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
15 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
30 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
60 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
3 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
6 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
12 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
24 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
48 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
72 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
96 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.120	0.157	0.187	0.227	0.257	0.288	0.320	0.362
10 min	0.173	0.225	0.268	0.325	0.369	0.413	0.459	0.519
15 min	0.208	0.273	0.325	0.393	0.446	0.500	0.554	0.628
30 min	0.286	0.375	0.445	0.540	0.612	0.686	0.761	0.862
60 min	0.396	0.518	0.616	0.747	0.848	0.949	1.053	1.193
3 hour	0.645	0.797	0.923	1.097	1.235	1.379	1.531	1.746
6 hour	0.879	1.067	1.224	1.443	1.617	1.799	1.993	2.267
12 hour	1.168	1.426	1.639	1.932	2.161	2.396	2.643	2.982
24 hour	1.603	1.983	2.292	2.711	3.032	3.358	3.694	4.149
48 hour	2.038	2.519	2.905	3.418	3.806	4.195	4.590	5.117
72 hour	2.345	2.893	3.330	3.909	4.342	4.775	5.211	5.791
96 hour	2.581	3.184	3.660	4.289	4.758	5.224	5.691	6.310

Attachment 4- F. ITR Comment Forms for Mosher Slough HEC-HMS Modeling

Lower San Joaquin River Feasibility Study

HYDROLOGIC MODELING REVIEW – MOSHER SLOUGH WATERSHED

Task Order 6 – LSJRFS Work-In-Kind Hydrology

Reviewer: Domenichelli & Associates
Review Date: 9-14-10
PBI Response Date: 9-28-10
D&A Backcheck: 10-8-10

Backcheck Comments:

1. No additional comments on the revised memo and model. All comments have been addressed adequately.

2. A general observation about the Mosher Slough project is that interior drainage behind the levees (and floodwalls) was not thoroughly address in the original SJAFCA project. For the 100-yr event, the pumps will not keep up and significant storage occurs behind the levees (approx 50 ac-ft at Don Ave PS). May need to consider analyzing interior drainage and mapping potential interior floodplains later in the process.

   PBI Response: Agreed. Additional analysis would be required to assess the dynamics of interior drainage within Mosher Slough subbasins.

Lower San Joaquin River Feasibility Study

HYDROLOGIC MODELING REVIEW – MOSHER SLOUGH WATERSHED

Task Order 6 – LSJRFS Work-In-Kind Hydrology

Reviewer: Domenichelli & Associates
Review Date: 9-14-10
PBI Response Date: 9-28-10

Note to Reviewer: Two subbasins (ATLAS & TCREEKS) were added to the PBI Model just west of Interstate-5. This extends the model to Mosher Slough's confluence with Bear Creek. All methodology for subbasin parameterization remained consistent with the rest of the subbasins.

Memorandum Comments:

1. *Page 7, Paragraph 1- Complete the Sentence*

PBI Response: Agreed.

2. *Section 4.4.2 – For a 72-hour event, the detention basins will likely fill requiring pumping back into the slough with the pump station capacities designed for the project. Be sure that both are modeled with pumping.*

PBI Response: Agreed. See Section 4.4.2.

3. *For more accurate modeling of the detention basins inflow and outflow, the final modeler should consider using the HEC-RAS Un-Steady State modeling routine. Side weirs and pumping rate information can be more accurately input into the RAS model than what is used in the HEC-HMS model.*

PBI Response: Agreed. The features in HEC-RAS can be used to perform analyses that cannot be completed in HMS. A HEC-RAS analysis is beyond the scope of this study. However, the hydraulics analysis for the LSJRFS planned for 2011 will include HEC-RAS unsteady modeling.

4. *Table 2, Consider providing a column for routing reach description (ie..Basin B7 to B6).*

PBI Response: Agreed.

6. *Section 4.4.6, Loss Rates- The initial loss rate is conservative (low) per last paragraph but the constant rate is at the mid to upper limit. Is this due to calibration? Why not be consistent with the Bear Creek rates. Again, may want to include a soils map somewhere for confirmation.*

PBI Response: The methodology for selecting constant loss rates has been modified. See Section 4.4.6.

7. *Section 4.5 Calibration- Calibrating to 790cfs at the location indicated provides results that are much higher than the SJAFCA model. Regional Equations are not very accurate and we would not recommend using them as a means for calibration. A translation of the parameters from the Bear Creek calibration seems more appropriate.*

PBI Response: Agreed. See Section 4.5.

Model Comments:

8. *Looking at the model results using the loss rates in Section 4.4.6 we would not expect the results to be almost double the SJAFCA model results for the sheds 1104 and 1105. We cannot find the reason for this large discrepancy. Please provide an explanation. Without the original SJAFCA model we cannot compare the input.*

PBI Response: With the model no longer being calibrated to 1/100 AEP peak flows calculated with the regression equation, combined peak flows coming from subbasins 1104 and 1105 are now only ~45% higher than the SJAFCA HEC-1 Model. This 45% difference is due not only to the different loss rate methodology used in the PBI Model, but also because subbasin areas and lag times were re-calculated for the PBI Model and differ from the SJAFCA HEC-1 Model.

9. *As stated in Comment #2 using the longer (72-hr) duration event and higher peak flows (per comment 8) and volumes, the volume of flow and timing into the detention basin must be checked. For the 100-yr event, Detention Basin #1 may fill at such a time that more than 230cfs will pass down Mosher Slough. Confirm that there is adequate storage to maintain the peak flow passing at 230cfs or if not how the peak 100-yr flows downstream will be impacted.*

PBI Response: Agreed. See Section 4.4.

Attachment 4- G. SPK Comment Forms for Mosher Slough HEC-HMS Modeling

Corps of Engineers, Hydrology Section, Review of Mosher Slough HEC-1 to HEC-HMS model conversion and preliminary report.

12 November 2010

Steven F. Holmstrom, P.E.

The Technical Memorandum for the Lower San Joaquin River Feasibility Study Mosher Slough HEC-HMS modeling DRAFT hydrology report has been reviewed and the following comments are provided.

12. In section 4.3 Design Storms, the fourth paragraph states that two storm centerings will be analyzed. A third storm centering will be required to compute the flow above the New Hogan Dam for the 8 design storm frequencies.

    PBI Response: It is PBI's understanding that the third storm centering above New Hogan Dam will only be analyzed for the Calaveras River watershed. No changes were made to this section.

13. In Section 4.4.2 Reservoirs and Pumps, it must be made clear that the pumps discharge into the receiving channel above the highest stage to be expected so that there is independence between the exterior and interior areas. If that is not the case then a coincidence analysis must be performed to determine the modified interior pond stage-frequency curve considering the exterior-interior stage conditions. This is explained in EM1110-2-1413, Hydrologic Analysis of Interior Areas.

    PBI Response: Section 4.4.2 has been updated.

14. In figure 5, Mosher Slough subbasin flowpaths, the blue line representing the flowpath should exit into Bear Creek as the entire subarea is diverted for all flow frequencies.

    PBI Response: The flowpath for the Atlas Tract has been changed and now exits into Bear Creek. The lag time calculation has also been updated for this subbasin.

15. In section 4.6, Model Simulations, Existing Conditions, the statement is made that "any subbasin flows exceeding pump station capacities would result in temporary ponding within the subbasin." The study must be clear that this condition is not related to exterior stages in the receiving stream. Or a coincidence analysis must be performed to relate interior and exterior stages.

    PBI Response: Section 4.6 has been updated.

16. Subbasin 1103D must be added to the table in attachment A which compares the HEC1 and HMS results.

    PBI Response: Subbasin 1103D was left out of the 1998 HEC-1 report's results table (perhaps unintentionally) and therefore was not able to be compared to the HEC-HMS results.

    The results table originally included in Attachment A was intended to show that the conversion from the HEC-1 model to HEC-HMS went smoothly. The HEC-HMS results listed in that table were for a model that still used the old methodology from the 1998 HEC-1 study (curve numbers, etc). This table has now been excluded from PBI's report so as to avoid any confusion. Attachment A now includes a table of the parameters used in the 1998 SJAFCA HEC-1 model.

17. The HMS model transmitted with the report does not appear to match either the results from the 1998 study or the calibration done for the current study. The report must clarify the purpose and the state of the input parameters contained in the model supplied.

    PBI Response: The HMS model transmitted with the report is not expected to match the results from the 1998 study or from the calibration run.

    As previously mentioned, the results table originally included in Attachment A was meant to show that the conversion from the HEC-1 model to HEC-HMS went smoothly. The HEC-HMS results listed in that table were for a model that still used the old methodology from the 1998 HEC-1 study (curve numbers, etc). This table has now been excluded from PBI's report so as to avoid any confusion.

    Calibration flow results were not included in the Mosher Slough report. As mentioned in Section 4.5, this model was calibrated based on the loss rate adjustment factor determined in the Bear Creek calibration.

    The input parameters for the transmitted model are listed in Attachment C & Attachment D.

Attachment 5- A. Calaveras River Watershed Subbasin Parameters Used in the 1998 SJAFCA HEC-1 Model

COMBINED CALAVERAS RIVER/DIVERTING CANAL/MORMON SLOUGH HYDROLOGY

Parameter	Description	Basin											
		P10	P20	MS10	P60	P40	P30	P50	P70	MS20	MS30	DIVA0	DIVA1
DA,mi.^2	Drainage Area	4.66	20.1	4.71	4.22	3.36	7.39	5.89	3.47	2.6	1.6	5.2	2.2
XL, mi.	Watercourse Length	2.86	9.1	7.58	4.73	1.81	1.81	4.17	5.68	3.79	2.27	7	3.22
XLCA, mi.	Length from Centroid	1.28	5.31	3.41	2.84	1.16	1.16	1.99	3.31	1.89	1.23	3.59	1.33
S, ft./mi.	Subarea Slope	55.97	21.12	5.28	9.5	23.23	23.23	12	4.4	5.28	4.86	6.2	6.52
BN	Basin "n" Value	0.15	0.15	0.1	0.1	0.15	0.15	0.15	0.1	0.1	0.1	0.1	0.1
ARF, mi^2	Storm Area Reduction Factor	50	50	50	50	50	50	50	50	50	50	220	220
Peak, cfs	Peak Flow	850.7	2887.8	609.0	504.7	1250.5	2750.4	606.1	329.9	347.1	289.6	476.5	361.7
Lag, hrs.	Calculated Basin Lag Time	2.74	8.80	6.02	4.20	2.63	2.63	5.02	5.52	3.70	2.63	5.78	2.92
SCS Num.	NCRS Curve Number	84	84	70	70	84	84	84	84	76	82	70	70
S Curve		FH	FH	VD	VU	FH	FH	VU	VU	VU	VU	VU	VU

Parameter	Description	Basin											
		DIVA2	DIVA3	DIVB1	DIVB2	DIVB3	DIVB4	DIVB5	DIVB6	DIVB7	DIVC1	DIVC2	C10
DA,mi.^2	Drainage Area	4.35	2.6	4.8	3.5	3.2	2.4	6.9	10.1	2	3.78	1.47	7.85
XL, mi.	Watercourse Length	3.9	4.73	5.3	3.13	3.59	3.03	4.17	5.1	3.21	4.54	1.61	7.73
XLCA, mi.	Length from Centroid	2	2.84	2.46	1.6	1.85	1.61	2.08	2.65	2.08	2.08	1	2.88
S, ft./mi.	Subarea Slope	3.5	5.3	8.11	5.75	5.01	5.3	6.95	4.71	4.75	6.2	2.48	25.34
BN	Basin "n" Value	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.08
ARF, mi^2	Storm Area Reduction Factor	220	220	220	220	220	220	220	220	220	220	220	20
Peak, cfs	Peak Flow	527.8	283.0	565.0	534.2	434.3	364.5	904.9	1076.8	268.6	473.1	307.1	2309.0
Lag, hrs.	Calculated Basin Lag Time	4.13	4.69	4.28	3.18	3.63	3.19	3.77	4.81	3.67	3.98	2.42	3.38
SCS Num.	NCRS Curve Number	71	79	70	74	70	79	70	70	75	76	76	82
S Curve		VU	VU	VU	VU	VU	VU	VU	VU	VU	VU	VU	FH

Parameter	Description	Basin						
		C20	C30	C40	C50	C60	C70	C80
DA,mi.^2	Drainage Area	5.8	0.75	1.6	1.3	2.7	1.5	2.17
XL, mi.	Watercourse Length	4.3	1.7	2.27	2.27	3.4	2.08	2.6
XLCA, mi.	Length from Centroid	1.7	1	1.32	1.5	2	1.1	1.4
S, ft./mi.	Subarea Slope	24.41	5.9	4.4	4.4	2.9	4.7	3.88
BN	Basin "n" Value	0.08	0.08	0.08	0.08	0.08	0.08	0.08
ARF, mi^2	Storm Area Reduction Factor	20	20	20	20	20	20	83
Peak, cfs	Peak Flow	2378.3	208.1	340.4	264.6	403.7	438.7	524.2
Lag, hrs.	Calculated Basin Lag Time	2.23	1.68	2.20	2.31	3.25	1.96	2.42
SCS Num.	NCRS Curve Number	83	79	76	77	72	82	83
S Curve		FH	VU	VU	VU	VU	VD	VD

S Curve Designations:

FH	Foothill S-Curve
VU	Valley Undeveloped
VD	Valley Developed

1/13/98

Source: HDR, Final Technical Memorandum #1, Hydrologic Report prepared for San Joaquin Area Flood Control Agency, January 1998.

Attachment 5- B. Calaveras River Subbasin Characteristics

Basin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
BL10	72.63	0.15	14.81	700	142	7.83	37.69	10.99	FH	1.5	0.052	2	
CG10	21.35	0.15	9.65	1550	500	5.74	108.76	6.79	FH	1.5	0.059	5	
NH10	1.19	0.15	1.90	850	500	0.87	183.94	1.62	FH	1.5	0.065	2	
DUCK	9.76	0.15	4.21	430	200	2.35	54.63	4.02	FH	1.5	0.028	2	
MS10	4.09	0.2	6.90	123	81	3.81	6.08	11.80	VU	1.5	0.158	2	
P60	1.80	0.2	4.48	104	77	1.82	6.02	7.58	VU	1.5	0.114	2	
P20	19.85	0.2	9.82	320	101	5.23	22.30	11.89	FH	1.5	0.035	2	
P10	5.64	0.2	4.34	320	142	2.17	41.02	5.56	FH	1.5	0.033	2	
P50	10.62	0.2	5.63	100	75	1.80	4.44	8.72	VU	1.5	0.037	2	
P30	4.85	0.2	2.86	238	127	1.30	38.86	3.95	VU	1.5	0.024	2	
P40	4.05	0.2	2.91	224	105	0.75	40.90	3.19	VU	1.5	0.021	2	
MS20	3.10	0.2	4.32	81	51	2.42	6.94	8.11	VU	1.5	0.063	2	
P70	2.33	0.2	4.17	77	51	2.56	6.24	8.33	VU	1.5	0.021	2	
MS30	1.47	0.2	2.22	53	32	1.17	9.44	4.50	VU	1.5	0.021	2	
DIVA2	5.56	0.2	3.56	86	62	1.72	6.74	6.65	VU	1.5	0.096	2	
DIVA1	2.09	0.2	2.92	103	80	0.95	7.87	4.77	VU	1.5	0.136	2	16.0
DIVA3	3.96	0.15	5.17	69	30	2.57	7.54	6.55	VU	1.5	0.064	5	
DIVA0	3.37	0.2	5.82	73	34	2.60	6.70	9.39	VU	1.5	0.026	2	
DIVB6	6.87	0.2	4.36	70	46	2.04	5.51	7.96	VU	1.5	0.115	2	
DIVB5	6.85	0.2	3.42	98	65	1.48	9.65	5.78	VU	1.5	0.130	2	
DIVB2	3.85	0.2	4.31	96	65	1.92	7.19	7.36	VU	1.5	0.114	2	
DIVB1	3.44	0.2	3.28	115	86	1.59	8.84	5.93	VU	1.5	0.148	2	100.0
DIVB3	3.87	0.2	3.31	66	46	1.46	6.04	6.22	VU	1.5	0.100	2	
DIVB4	1.96	0.15	3.13	48	29	1.73	6.07	4.85	VU	1.5	0.047	5	
DIVB7	2.09	0.15	3.14	45	29	1.56	5.09	4.83	VU	1.5	0.063	5	
SANG	0.49	0.015	1.31	20	19	1.31	0.76	0.46	VD	1.5	0.045	60	92.2
C10	7.76	0.2	6.00	284	124	2.75	26.68	7.46	FH	1.5	0.031	0	
C20	7.11	0.2	6.08	195	95	3.26	16.45	8.77	VU	1.5	0.074	2	
C30	1.71	0.2	3.83	96	72	1.66	6.26	6.85	VU	1.5	0.128	2	

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Basin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
C40	2.19	0.2	2.57	74	56	1.16	7.01	5.02	VU	1.5	0.108	2	
C60	3.75	0.15	3.61	53	33	1.52	5.54	4.97	VU	1.5	0.092	5	
C50	1.63	0.2	1.99	59	48	0.95	5.52	4.42	VU	1.5	0.078	2	
C70	2.26	0.1	2.88	41	30	1.38	3.82	3.14	VU	1.5	0.022	15	
HOLM	1.73	0.02	1.91	30	26	0.69	2.10	0.46	VD	1.5	0.028	30	140.1
DIVC1	2.39	0.15	3.13	48	1	1.60	15.01	3.97	VU	1.5	0.060	5	
DIVC2	1.21	0.15	1.80	29	1	0.88	15.53	2.54	VD	1.5	0.041	15	
WLN	0.71	0.015	1.55	25	20	0.65	3.23	0.29	VD	1.5	0.028	60	47.5
WLS	0.36	0.015	1.28	17	16	0.42	0.78	0.30	VD	1.5	0.021	60	254.0
BCHI	1.27	0.015	1.67	20	15	0.52	3.00	0.28	VD	1.5	0.021	60	176.9
SUT	0.67	0.015	1.32	17	15	0.52	1.52	0.29	VD	1.5	0.021	60	54.1
BSTG	0.45	0.015	1.09	12	8	0.55	3.67	0.23	VD	1.5	0.023	60	132.7
RWLK	0.02	0.015	0.18	9	7	0.05	10.99	0.04	VD	1.5	0.158	60	10.5
MBRK	0.61	0.015	0.99	1	0	0.32	1.01	0.23	VD	1.5	0.058	60	6.0
PLYM	0.10	0.015	0.57	1	0	0.26	1.77	0.16	VD	1.5	0.080	60	121.3
HGTY	0.10	0.015	0.57	6	5	0.26	1.77	0.16	VD	1.5	0.109	60	6.2
BRES	0.42	0.02	1.44	1	0	0.61	0.70	0.49	VD	1.5	0.085	40	39.0
KIRK	0.05	0.015	0.31	3	1	0.17	6.41	0.08	VD	1.5	0.085	60	14.5
WISC	1.14	0.015	1.56	5	3	0.37	1.28	0.28	VD	1.5	0.078	60	21.7

Notes: FH = Foothill; VU = Valley Undeveloped; VD = Valley Developed

Attachment 5- C. Calaveras River Subbasin Soil Groups and Loss Rates

Soil Group:	A (0.35 in/hr)	B (0.2 in/hr)	C (0.1 in/hr)	D (0.025 in/hr)	Composite Loss Rate	Adjusted Loss Rate (Adjustment Factor = 0.85)
Subbasin	[sq. mi.]	[sq. mi.]	[sq. mi.]	[sq. mi.]	[in/hr]	[in/hr]
BL10	0.47	12.31	3.19	55.57	0.061	0.052
CG10	0.00	4.47	1.83	14.16	0.070	0.059
NH10	0.00	0.10	0.54	0.49	0.076	0.065
DUCK	0.00	0.07	0.93	8.68	0.034	0.028
MS10	0.00	3.47	0.28	0.16	0.186	0.158
P60	0.00	0.72	0.93	0.14	0.134	0.114
P20	0.00	1.72	0.10	17.89	0.041	0.035
P10	0.15	0.00	0.39	4.98	0.039	0.033
P50	0.00	0.86	0.65	9.10	0.044	0.037
P30	0.00	0.10	0.00	4.75	0.028	0.024
P40	0.00	0.00	0.00	4.05	0.025	0.021
MS20	0.00	0.11	1.68	1.20	0.074	0.063
P70	0.00	0.00	0.01	2.32	0.025	0.021
MS30	0.00	0.00	0.00	1.41	0.025	0.021
DIVA2	0.00	1.06	4.04	0.46	0.113	0.096
DIVA1	0.00	1.26	0.83	0.00	0.160	0.136
DIVA3	0.00	0.02	2.63	1.29	0.076	0.064
DIVA0	0.00	0.00	0.24	3.07	0.030	0.026
DIVB6	0.00	2.44	4.42	0.01	0.135	0.115
DIVB5	0.00	3.62	3.23	0.00	0.153	0.130
DIVB2	0.00	1.29	2.56	0.00	0.134	0.114
DIVB1	0.00	2.63	0.70	0.11	0.174	0.148
DIVB3	0.00	0.69	3.18	0.00	0.118	0.100
DIVB4	0.00	0.00	0.78	1.14	0.055	0.047
DIVB7	0.00	0.00	1.23	0.67	0.074	0.063
SANG	0.00	0.00	0.18	0.30	0.053	0.045
C10	0.00	0.04	1.08	6.39	0.037	0.031
C20	0.00	1.83	1.45	3.68	0.087	0.074
C30	0.00	0.82	0.79	0.00	0.151	0.128
C40	0.00	0.78	1.08	0.28	0.127	0.108
C60	0.00	1.54	0.50	1.66	0.108	0.092
C50	0.00	0.37	0.56	0.67	0.091	0.078
C70	0.00	0.00	0.02	2.19	0.026	0.022
HOLM	0.00	0.00	0.18	1.54	0.033	0.028
DIVC1	0.00	0.04	1.30	0.95	0.070	0.060
DIVC2	0.00	0.03	0.19	0.60	0.049	0.041
WLN	0.00	0.00	0.08	0.63	0.033	0.028
WLS	0.00	0.00	0.00	0.36	0.025	0.021
BCHI	0.00	0.00	0.00	1.27	0.025	0.021
SUT	0.00	0.00	0.00	0.65	0.025	0.021
BSTG	0.00	0.00	0.01	0.44	0.027	0.023
RWLK	0.00	0.02	0.00	0.00	0.186	0.158
MBRK	0.00	0.00	0.35	0.25	0.069	0.058
PLYM	0.00	0.00	0.09	0.01	0.094	0.080
HGTY	0.00	0.03	0.07	0.00	0.128	0.109
BRES	0.00	0.00	0.40	0.00	0.100	0.085
KIRK	0.00	0.00	0.04	0.00	0.100	0.085
WISC	0.00	0.00	0.96	0.12	0.092	0.078

Attachment 5- D. Calaveras River Depth-Duration-Frequency Tables

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SFS

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.134	0.176	0.209	0.254	0.288	0.322	0.358	0.405
10 min	0.193	0.252	0.300	0.364	0.413	0.462	0.513	0.581
15 min	0.233	0.305	0.363	0.440	0.499	0.559	0.620	0.702
30 min	0.320	0.419	0.498	0.604	0.685	0.767	0.851	0.964
60 min	0.443	0.579	0.689	0.836	0.948	1.062	1.178	1.335
3 hour	0.722	0.891	1.032	1.227	1.381	1.542	1.713	1.953
6 hour	0.983	1.193	1.369	1.614	1.809	2.012	2.229	2.536
12 hour	1.307	1.595	1.833	2.161	2.417	2.680	2.956	3.336
24 hour	1.793	2.218	2.564	3.032	3.392	3.756	4.132	4.641
48 hour	2.280	2.818	3.249	3.823	4.257	4.692	5.134	5.724
72 hour	2.623	3.236	3.725	4.372	4.857	5.341	5.829	6.478
96 hour	2.887	3.561	4.094	4.797	5.322	5.843	6.366	7.058

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
10 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
15 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
30 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
60 min	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
3 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
6 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
12 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
24 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
48 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
72 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894
96 hour	0.894	0.894	0.894	0.894	0.894	0.894	0.894	0.894

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.120	0.157	0.187	0.227	0.257	0.288	0.320	0.362
10 min	0.173	0.225	0.268	0.325	0.369	0.413	0.459	0.519
15 min	0.208	0.273	0.325	0.393	0.446	0.500	0.554	0.628
30 min	0.286	0.375	0.445	0.540	0.612	0.686	0.761	0.862
60 min	0.396	0.518	0.616	0.747	0.848	0.949	1.053	1.193
3 hour	0.645	0.797	0.923	1.097	1.235	1.379	1.531	1.746
6 hour	0.879	1.067	1.224	1.443	1.617	1.799	1.993	2.267
12 hour	1.168	1.426	1.639	1.932	2.161	2.396	2.643	2.982
24 hour	1.603	1.983	2.292	2.711	3.032	3.358	3.694	4.149
48 hour	2.038	2.519	2.905	3.418	3.806	4.195	4.590	5.117
72 hour	2.345	2.893	3.330	3.909	4.342	4.775	5.211	5.791
96 hour	2.581	3.184	3.660	4.289	4.758	5.224	5.691	6.310

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SFS

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.134	0.176	0.209	0.254	0.288	0.322	0.358	0.405
10 min	0.193	0.252	0.300	0.364	0.413	0.462	0.513	0.581
15 min	0.233	0.305	0.363	0.440	0.499	0.559	0.620	0.702
30 min	0.320	0.419	0.498	0.604	0.685	0.767	0.851	0.964
60 min	0.443	0.579	0.689	0.836	0.948	1.062	1.178	1.335
3 hour	0.722	0.891	1.032	1.227	1.381	1.542	1.713	1.953
6 hour	0.983	1.193	1.369	1.614	1.809	2.012	2.229	2.536
12 hour	1.307	1.595	1.833	2.161	2.417	2.680	2.956	3.336
24 hour	1.793	2.218	2.564	3.032	3.392	3.756	4.132	4.641
48 hour	2.280	2.818	3.249	3.823	4.257	4.692	5.134	5.724
72 hour	2.623	3.236	3.725	4.372	4.857	5.341	5.829	6.478
96 hour	2.887	3.561	4.094	4.797	5.322	5.843	6.366	7.058

Bellota Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.497	0.510	0.507	0.504	0.505	0.504	0.504	0.502
10 min	0.503	0.504	0.507	0.504	0.504	0.503	0.502	0.500
15 min	0.503	0.504	0.505	0.506	0.505	0.504	0.502	0.500
30 min	0.502	0.505	0.506	0.504	0.505	0.503	0.501	0.498
60 min	0.504	0.507	0.508	0.507	0.506	0.505	0.504	0.501
3 hour	0.501	0.500	0.499	0.498	0.496	0.495	0.493	0.492
6 hour	0.496	0.494	0.493	0.491	0.490	0.489	0.488	0.487
12 hour	0.491	0.489	0.487	0.485	0.485	0.484	0.483	0.482
24 hour	0.486	0.484	0.483	0.482	0.481	0.481	0.481	0.481
48 hour	0.482	0.480	0.479	0.477	0.477	0.477	0.477	0.477
72 hour	0.480	0.477	0.475	0.474	0.474	0.474	0.473	0.474
96 hour	0.479	0.475	0.474	0.473	0.472	0.472	0.472	0.473

Bellota Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.067	0.090	0.106	0.128	0.145	0.162	0.180	0.203
10 min	0.097	0.127	0.152	0.183	0.208	0.232	0.258	0.291
15 min	0.117	0.154	0.183	0.223	0.252	0.282	0.311	0.351
30 min	0.161	0.212	0.252	0.304	0.346	0.386	0.426	0.480
60 min	0.223	0.294	0.350	0.424	0.480	0.536	0.594	0.669
3 hour	0.362	0.446	0.515	0.611	0.685	0.763	0.845	0.961
6 hour	0.488	0.589	0.675	0.792	0.886	0.984	1.088	1.235
12 hour	0.642	0.780	0.893	1.048	1.172	1.297	1.428	1.608
24 hour	0.871	1.074	1.238	1.461	1.632	1.807	1.987	2.232
48 hour	1.099	1.353	1.556	1.824	2.031	2.238	2.449	2.730
72 hour	1.259	1.544	1.769	2.072	2.302	2.532	2.757	3.071
96 hour	1.383	1.691	1.941	2.269	2.512	2.758	3.005	3.338

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SFS

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.134	0.176	0.209	0.254	0.288	0.322	0.358	0.405
10 min	0.193	0.252	0.300	0.364	0.413	0.462	0.513	0.581
15 min	0.233	0.305	0.363	0.440	0.499	0.559	0.620	0.702
30 min	0.320	0.419	0.498	0.604	0.685	0.767	0.851	0.964
60 min	0.443	0.579	0.689	0.836	0.948	1.062	1.178	1.335
3 hour	0.722	0.891	1.032	1.227	1.381	1.542	1.713	1.953
6 hour	0.983	1.193	1.369	1.614	1.809	2.012	2.229	2.536
12 hour	1.307	1.595	1.833	2.161	2.417	2.680	2.956	3.336
24 hour	1.793	2.218	2.564	3.032	3.392	3.756	4.132	4.641
48 hour	2.280	2.818	3.249	3.823	4.257	4.692	5.134	5.724
72 hour	2.623	3.236	3.725	4.372	4.857	5.341	5.829	6.478
96 hour	2.887	3.561	4.094	4.797	5.322	5.843	6.366	7.058

Above New Hogan Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.497	0.510	0.507	0.504	0.505	0.504	0.504	0.502
10 min	0.503	0.504	0.507	0.504	0.504	0.503	0.502	0.500
15 min	0.503	0.504	0.505	0.506	0.505	0.504	0.502	0.500
30 min	0.502	0.505	0.506	0.504	0.505	0.503	0.501	0.498
60 min	0.504	0.507	0.508	0.507	0.506	0.505	0.504	0.501
3 hour	0.501	0.500	0.499	0.498	0.496	0.495	0.493	0.492
6 hour	0.496	0.494	0.493	0.491	0.490	0.489	0.488	0.487
12 hour	0.491	0.489	0.487	0.485	0.485	0.484	0.483	0.482
24 hour	0.486	0.484	0.483	0.482	0.481	0.481	0.481	0.481
48 hour	0.482	0.480	0.479	0.477	0.477	0.477	0.477	0.477
72 hour	0.480	0.477	0.475	0.474	0.474	0.474	0.473	0.474
96 hour	0.479	0.475	0.474	0.473	0.472	0.472	0.472	0.473

Above New Hogan Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.067	0.090	0.106	0.128	0.145	0.162	0.180	0.203
10 min	0.097	0.127	0.152	0.183	0.208	0.232	0.258	0.291
15 min	0.117	0.154	0.183	0.223	0.252	0.282	0.311	0.351
30 min	0.161	0.212	0.252	0.304	0.346	0.386	0.426	0.480
60 min	0.223	0.294	0.350	0.424	0.480	0.536	0.594	0.669
3 hour	0.362	0.446	0.515	0.611	0.685	0.763	0.845	0.961
6 hour	0.488	0.589	0.675	0.792	0.886	0.984	1.088	1.235
12 hour	0.642	0.780	0.893	1.048	1.172	1.297	1.428	1.608
24 hour	0.871	1.074	1.238	1.461	1.632	1.807	1.987	2.232
48 hour	1.099	1.353	1.556	1.824	2.031	2.238	2.449	2.730
72 hour	1.259	1.544	1.769	2.072	2.302	2.532	2.757	3.071
96 hour	1.383	1.691	1.941	2.269	2.512	2.758	3.005	3.338

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SFS

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.134	0.176	0.209	0.254	0.288	0.322	0.358	0.405
10 min	0.193	0.252	0.300	0.364	0.413	0.462	0.513	0.581
15 min	0.233	0.305	0.363	0.440	0.499	0.559	0.620	0.702
30 min	0.320	0.419	0.498	0.604	0.685	0.767	0.851	0.964
60 min	0.443	0.579	0.689	0.836	0.948	1.062	1.178	1.335
3 hour	0.722	0.891	1.032	1.227	1.381	1.542	1.713	1.953
6 hour	0.983	1.193	1.369	1.614	1.809	2.012	2.229	2.536
12 hour	1.307	1.595	1.833	2.161	2.417	2.680	2.956	3.336
24 hour	1.793	2.218	2.564	3.032	3.392	3.756	4.132	4.641
48 hour	2.280	2.818	3.249	3.823	4.257	4.692	5.134	5.724
72 hour	2.623	3.236	3.725	4.372	4.857	5.341	5.829	6.478
96 hour	2.887	3.561	4.094	4.797	5.322	5.843	6.366	7.058

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.497	0.510	0.507	0.504	0.505	0.504	0.504	0.502
10 min	0.503	0.504	0.507	0.504	0.504	0.503	0.502	0.500
15 min	0.503	0.504	0.505	0.506	0.505	0.504	0.502	0.500
30 min	0.502	0.505	0.506	0.504	0.505	0.503	0.501	0.498
60 min	0.504	0.507	0.508	0.507	0.506	0.505	0.504	0.501
3 hour	0.501	0.500	0.499	0.498	0.496	0.495	0.493	0.492
6 hour	0.496	0.494	0.493	0.491	0.490	0.489	0.488	0.487
12 hour	0.491	0.489	0.487	0.485	0.485	0.484	0.483	0.482
24 hour	0.486	0.484	0.483	0.482	0.481	0.481	0.481	0.481
48 hour	0.482	0.480	0.479	0.477	0.477	0.477	0.477	0.477
72 hour	0.480	0.477	0.475	0.474	0.474	0.474	0.473	0.474
96 hour	0.479	0.475	0.474	0.473	0.472	0.472	0.472	0.473

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.067	0.090	0.106	0.128	0.145	0.162	0.180	0.203
10 min	0.097	0.127	0.152	0.183	0.208	0.232	0.258	0.291
15 min	0.117	0.154	0.183	0.223	0.252	0.282	0.311	0.351
30 min	0.161	0.212	0.252	0.304	0.346	0.386	0.426	0.480
60 min	0.223	0.294	0.350	0.424	0.480	0.536	0.594	0.669
3 hour	0.362	0.446	0.515	0.611	0.685	0.763	0.845	0.961
6 hour	0.488	0.589	0.675	0.792	0.886	0.984	1.088	1.235
12 hour	0.642	0.780	0.893	1.048	1.172	1.297	1.428	1.608
24 hour	0.871	1.074	1.238	1.461	1.632	1.807	1.987	2.232
48 hour	1.099	1.353	1.556	1.824	2.031	2.238	2.449	2.730
72 hour	1.259	1.544	1.769	2.072	2.302	2.532	2.757	3.071
96 hour	1.383	1.691	1.941	2.269	2.512	2.758	3.005	3.338

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SCK

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.171	0.204	0.249	0.284	0.319	0.356	0.408
10 min	0.187	0.245	0.292	0.357	0.406	0.458	0.511	0.584
15 min	0.227	0.296	0.353	0.431	0.491	0.553	0.618	0.707
30 min	0.311	0.407	0.486	0.593	0.675	0.760	0.849	0.971
60 min	0.430	0.563	0.671	0.819	0.934	1.051	1.174	1.343
3 hour	0.674	0.837	0.977	1.180	1.346	1.524	1.718	2.000
6 hour	0.893	1.097	1.274	1.529	1.740	1.967	2.215	2.576
12 hour	1.175	1.463	1.703	2.041	2.309	2.589	2.886	3.302
24 hour	1.621	2.050	2.397	2.868	3.229	3.595	3.972	4.482
48 hour	2.014	2.529	2.945	3.503	3.929	4.358	4.797	5.388
72 hour	2.273	2.839	3.295	3.908	4.375	4.847	5.329	5.979
96 hour	2.490	3.101	3.593	4.253	4.754	5.258	5.773	6.465

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.768	0.766	0.766	0.768	0.767	0.768	0.768	0.768
10 min	0.768	0.768	0.767	0.768	0.768	0.768	0.768	0.768
15 min	0.768	0.768	0.767	0.767	0.767	0.767	0.768	0.769
30 min	0.767	0.767	0.767	0.768	0.768	0.768	0.768	0.769
60 min	0.767	0.767	0.767	0.767	0.767	0.768	0.768	0.769
3 hour	0.765	0.766	0.766	0.766	0.767	0.768	0.769	0.769
6 hour	0.764	0.764	0.765	0.766	0.766	0.767	0.768	0.769
12 hour	0.763	0.764	0.765	0.766	0.766	0.767	0.767	0.768
24 hour	0.763	0.764	0.765	0.766	0.766	0.766	0.766	0.767
48 hour	0.762	0.763	0.763	0.764	0.764	0.765	0.765	0.765
72 hour	0.761	0.762	0.762	0.763	0.763	0.764	0.764	0.764
96 hour	0.761	0.761	0.762	0.762	0.763	0.763	0.763	0.764

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.101	0.131	0.156	0.191	0.218	0.245	0.273	0.313
10 min	0.144	0.188	0.224	0.274	0.312	0.352	0.392	0.449
15 min	0.174	0.227	0.271	0.331	0.377	0.424	0.475	0.544
30 min	0.239	0.312	0.373	0.455	0.518	0.584	0.652	0.747
60 min	0.330	0.432	0.515	0.628	0.716	0.807	0.902	1.033
3 hour	0.516	0.641	0.748	0.904	1.032	1.170	1.321	1.538
6 hour	0.682	0.838	0.975	1.171	1.333	1.509	1.701	1.981
12 hour	0.897	1.118	1.303	1.563	1.769	1.986	2.214	2.536
24 hour	1.237	1.566	1.834	2.197	2.473	2.754	3.043	3.438
48 hour	1.535	1.930	2.247	2.676	3.002	3.334	3.670	4.122
72 hour	1.730	2.163	2.511	2.982	3.338	3.703	4.071	4.568
96 hour	1.895	2.360	2.738	3.241	3.627	4.012	4.405	4.939

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SCK

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.171	0.204	0.249	0.284	0.319	0.356	0.408
10 min	0.187	0.245	0.292	0.357	0.406	0.458	0.511	0.584
15 min	0.227	0.296	0.353	0.431	0.491	0.553	0.618	0.707
30 min	0.311	0.407	0.486	0.593	0.675	0.760	0.849	0.971
60 min	0.430	0.563	0.671	0.819	0.934	1.051	1.174	1.343
3 hour	0.674	0.837	0.977	1.180	1.346	1.524	1.718	2.000
6 hour	0.893	1.097	1.274	1.529	1.740	1.967	2.215	2.576
12 hour	1.175	1.463	1.703	2.041	2.309	2.589	2.886	3.302
24 hour	1.621	2.050	2.397	2.868	3.229	3.595	3.972	4.482
48 hour	2.014	2.529	2.945	3.503	3.929	4.358	4.797	5.388
72 hour	2.273	2.839	3.295	3.908	4.375	4.847	5.329	5.979
96 hour	2.490	3.101	3.593	4.253	4.754	5.258	5.773	6.465

Bellota Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.500	0.506	0.503	0.508	0.503	0.507	0.508	0.505
10 min	0.506	0.508	0.506	0.508	0.508	0.506	0.505	0.503
15 min	0.507	0.508	0.505	0.507	0.506	0.505	0.506	0.505
30 min	0.501	0.505	0.506	0.505	0.506	0.505	0.504	0.503
60 min	0.504	0.506	0.508	0.508	0.507	0.507	0.506	0.505
3 hour	0.492	0.493	0.493	0.493	0.494	0.495	0.496	0.498
6 hour	0.481	0.482	0.481	0.483	0.484	0.486	0.487	0.491
12 hour	0.473	0.473	0.474	0.475	0.476	0.477	0.479	0.480
24 hour	0.467	0.469	0.470	0.471	0.472	0.473	0.473	0.474
48 hour	0.457	0.458	0.458	0.459	0.460	0.461	0.462	0.464
72 hour	0.450	0.449	0.448	0.450	0.451	0.452	0.454	0.456
96 hour	0.447	0.445	0.445	0.446	0.447	0.449	0.450	0.453

Bellota Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.066	0.087	0.103	0.126	0.143	0.162	0.181	0.206
10 min	0.095	0.124	0.148	0.181	0.206	0.232	0.258	0.294
15 min	0.115	0.150	0.178	0.219	0.248	0.279	0.313	0.357
30 min	0.156	0.206	0.246	0.299	0.342	0.384	0.428	0.488
60 min	0.217	0.285	0.341	0.416	0.474	0.533	0.594	0.678
3 hour	0.332	0.413	0.482	0.582	0.665	0.754	0.852	0.996
6 hour	0.430	0.529	0.613	0.739	0.842	0.956	1.079	1.265
12 hour	0.556	0.692	0.807	0.969	1.099	1.235	1.382	1.585
24 hour	0.757	0.961	1.127	1.351	1.524	1.700	1.879	2.124
48 hour	0.920	1.158	1.349	1.608	1.807	2.009	2.216	2.500
72 hour	1.023	1.275	1.476	1.759	1.973	2.191	2.419	2.726
96 hour	1.113	1.380	1.599	1.897	2.125	2.361	2.598	2.929

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SCK

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.171	0.204	0.249	0.284	0.319	0.356	0.408
10 min	0.187	0.245	0.292	0.357	0.406	0.458	0.511	0.584
15 min	0.227	0.296	0.353	0.431	0.491	0.553	0.618	0.707
30 min	0.311	0.407	0.486	0.593	0.675	0.760	0.849	0.971
60 min	0.430	0.563	0.671	0.819	0.934	1.051	1.174	1.343
3 hour	0.674	0.837	0.977	1.180	1.346	1.524	1.718	2.000
6 hour	0.893	1.097	1.274	1.529	1.740	1.967	2.215	2.576
12 hour	1.175	1.463	1.703	2.041	2.309	2.589	2.886	3.302
24 hour	1.621	2.050	2.397	2.868	3.229	3.595	3.972	4.482
48 hour	2.014	2.529	2.945	3.503	3.929	4.358	4.797	5.388
72 hour	2.273	2.839	3.295	3.908	4.375	4.847	5.329	5.979
96 hour	2.490	3.101	3.593	4.253	4.754	5.258	5.773	6.465

Above New Hogan Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.500	0.506	0.503	0.508	0.503	0.507	0.508	0.505
10 min	0.506	0.508	0.506	0.508	0.508	0.506	0.505	0.503
15 min	0.507	0.508	0.505	0.507	0.506	0.505	0.506	0.505
30 min	0.501	0.505	0.506	0.505	0.506	0.505	0.504	0.503
60 min	0.504	0.506	0.508	0.508	0.507	0.507	0.506	0.505
3 hour	0.492	0.493	0.493	0.493	0.494	0.495	0.496	0.498
6 hour	0.481	0.482	0.481	0.483	0.484	0.486	0.487	0.491
12 hour	0.473	0.473	0.474	0.475	0.476	0.477	0.479	0.480
24 hour	0.467	0.469	0.470	0.471	0.472	0.473	0.473	0.474
48 hour	0.457	0.458	0.458	0.459	0.460	0.461	0.462	0.464
72 hour	0.450	0.449	0.448	0.450	0.451	0.452	0.454	0.456
96 hour	0.447	0.445	0.445	0.446	0.447	0.449	0.450	0.453

Above New Hogan Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.066	0.087	0.103	0.126	0.143	0.162	0.181	0.206
10 min	0.095	0.124	0.148	0.181	0.206	0.232	0.258	0.294
15 min	0.115	0.150	0.178	0.219	0.248	0.279	0.313	0.357
30 min	0.156	0.206	0.246	0.299	0.342	0.384	0.428	0.488
60 min	0.217	0.285	0.341	0.416	0.474	0.533	0.594	0.678
3 hour	0.332	0.413	0.482	0.582	0.665	0.754	0.852	0.996
6 hour	0.430	0.529	0.613	0.739	0.842	0.956	1.079	1.265
12 hour	0.556	0.692	0.807	0.969	1.099	1.235	1.382	1.585
24 hour	0.757	0.961	1.127	1.351	1.524	1.700	1.879	2.124
48 hour	0.920	1.158	1.349	1.608	1.807	2.009	2.216	2.500
72 hour	1.023	1.275	1.476	1.759	1.973	2.191	2.419	2.726
96 hour	1.113	1.380	1.599	1.897	2.125	2.361	2.598	2.929

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SCK

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.171	0.204	0.249	0.284	0.319	0.356	0.408
10 min	0.187	0.245	0.292	0.357	0.406	0.458	0.511	0.584
15 min	0.227	0.296	0.353	0.431	0.491	0.553	0.618	0.707
30 min	0.311	0.407	0.486	0.593	0.675	0.760	0.849	0.971
60 min	0.430	0.563	0.671	0.819	0.934	1.051	1.174	1.343
3 hour	0.674	0.837	0.977	1.180	1.346	1.524	1.718	2.000
6 hour	0.893	1.097	1.274	1.529	1.740	1.967	2.215	2.576
12 hour	1.175	1.463	1.703	2.041	2.309	2.589	2.886	3.302
24 hour	1.621	2.050	2.397	2.868	3.229	3.595	3.972	4.482
48 hour	2.014	2.529	2.945	3.503	3.929	4.358	4.797	5.388
72 hour	2.273	2.839	3.295	3.908	4.375	4.847	5.329	5.979
96 hour	2.490	3.101	3.593	4.253	4.754	5.258	5.773	6.465

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.500	0.506	0.503	0.508	0.503	0.507	0.508	0.505
10 min	0.506	0.508	0.506	0.508	0.508	0.506	0.505	0.503
15 min	0.507	0.508	0.505	0.507	0.506	0.505	0.506	0.505
30 min	0.501	0.505	0.506	0.505	0.506	0.505	0.504	0.503
60 min	0.504	0.506	0.508	0.508	0.507	0.507	0.506	0.505
3 hour	0.492	0.493	0.493	0.493	0.494	0.495	0.496	0.498
6 hour	0.481	0.482	0.481	0.483	0.484	0.486	0.487	0.491
12 hour	0.473	0.473	0.474	0.475	0.476	0.477	0.479	0.480
24 hour	0.467	0.469	0.470	0.471	0.472	0.473	0.473	0.474
48 hour	0.457	0.458	0.458	0.459	0.460	0.461	0.462	0.464
72 hour	0.450	0.449	0.448	0.450	0.451	0.452	0.454	0.456
96 hour	0.447	0.445	0.445	0.446	0.447	0.449	0.450	0.453

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.066	0.087	0.103	0.126	0.143	0.162	0.181	0.206
10 min	0.095	0.124	0.148	0.181	0.206	0.232	0.258	0.294
15 min	0.115	0.150	0.178	0.219	0.248	0.279	0.313	0.357
30 min	0.156	0.206	0.246	0.299	0.342	0.384	0.428	0.488
60 min	0.217	0.285	0.341	0.416	0.474	0.533	0.594	0.678
3 hour	0.332	0.413	0.482	0.582	0.665	0.754	0.852	0.996
6 hour	0.430	0.529	0.613	0.739	0.842	0.956	1.079	1.265
12 hour	0.556	0.692	0.807	0.969	1.099	1.235	1.382	1.585
24 hour	0.757	0.961	1.127	1.351	1.524	1.700	1.879	2.124
48 hour	0.920	1.158	1.349	1.608	1.807	2.009	2.216	2.500
72 hour	1.023	1.275	1.476	1.759	1.973	2.191	2.419	2.726
96 hour	1.113	1.380	1.599	1.897	2.125	2.361	2.598	2.929

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FRM

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.127	0.174	0.211	0.260	0.298	0.335	0.373	0.424
10 min	0.182	0.249	0.302	0.373	0.427	0.480	0.534	0.607
15 min	0.220	0.301	0.365	0.451	0.516	0.581	0.646	0.734
30 min	0.306	0.418	0.508	0.627	0.717	0.808	0.899	1.021
60 min	0.415	0.567	0.689	0.850	0.972	1.094	1.218	1.384
3 hour	0.664	0.874	1.049	1.292	1.484	1.683	1.892	2.184
6 hour	0.889	1.163	1.393	1.714	1.968	2.234	2.515	2.908
12 hour	1.188	1.585	1.910	2.357	2.704	3.059	3.428	3.931
24 hour	1.619	2.190	2.648	3.260	3.722	4.184	4.654	5.280
48 hour	2.005	2.665	3.186	3.870	4.378	4.880	5.381	6.038
72 hour	2.286	3.004	3.566	4.298	4.839	5.369	5.896	6.581
96 hour	2.468	3.217	3.802	4.561	5.118	5.663	6.203	6.902

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.621	0.620	0.626	0.626	0.627	0.626	0.625	0.624
10 min	0.616	0.622	0.625	0.625	0.627	0.626	0.625	0.626
15 min	0.618	0.623	0.626	0.626	0.627	0.626	0.626	0.625
30 min	0.619	0.624	0.626	0.628	0.627	0.628	0.627	0.626
60 min	0.616	0.623	0.625	0.626	0.626	0.626	0.626	0.625
3 hour	0.616	0.622	0.626	0.628	0.629	0.630	0.630	0.630
6 hour	0.616	0.622	0.626	0.628	0.630	0.631	0.632	0.633
12 hour	0.616	0.624	0.628	0.631	0.633	0.634	0.635	0.636
24 hour	0.615	0.623	0.627	0.630	0.632	0.633	0.634	0.635
48 hour	0.613	0.620	0.624	0.626	0.628	0.628	0.629	0.629
72 hour	0.614	0.619	0.622	0.624	0.625	0.626	0.627	0.627
96 hour	0.612	0.617	0.619	0.621	0.622	0.623	0.623	0.623

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.079	0.108	0.132	0.163	0.187	0.210	0.233	0.265
10 min	0.112	0.155	0.189	0.233	0.268	0.300	0.334	0.380
15 min	0.136	0.188	0.228	0.282	0.324	0.364	0.404	0.459
30 min	0.189	0.261	0.318	0.394	0.450	0.507	0.564	0.639
60 min	0.256	0.353	0.431	0.532	0.608	0.685	0.762	0.865
3 hour	0.409	0.544	0.657	0.811	0.933	1.060	1.192	1.376
6 hour	0.548	0.723	0.872	1.076	1.240	1.410	1.589	1.841
12 hour	0.732	0.989	1.199	1.487	1.712	1.939	2.177	2.500
24 hour	0.996	1.364	1.660	2.054	2.352	2.648	2.951	3.353
48 hour	1.229	1.652	1.988	2.423	2.749	3.065	3.385	3.798
72 hour	1.404	1.859	2.218	2.682	3.024	3.361	3.697	4.126
96 hour	1.510	1.985	2.353	2.832	3.183	3.528	3.864	4.300

CALAVERAS RIVER WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: FRM

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.127	0.174	0.211	0.260	0.298	0.335	0.373	0.424
10 min	0.182	0.249	0.302	0.373	0.427	0.480	0.534	0.607
15 min	0.220	0.301	0.365	0.451	0.516	0.581	0.646	0.734
30 min	0.306	0.418	0.508	0.627	0.717	0.808	0.899	1.021
60 min	0.415	0.567	0.689	0.850	0.972	1.094	1.218	1.384
3 hour	0.664	0.874	1.049	1.292	1.484	1.683	1.892	2.184
6 hour	0.889	1.163	1.393	1.714	1.968	2.234	2.515	2.908
12 hour	1.188	1.585	1.910	2.357	2.704	3.059	3.428	3.931
24 hour	1.619	2.190	2.648	3.260	3.722	4.184	4.654	5.280
48 hour	2.005	2.665	3.186	3.870	4.378	4.880	5.381	6.038
72 hour	2.286	3.004	3.566	4.298	4.839	5.369	5.896	6.581
96 hour	2.468	3.217	3.802	4.561	5.118	5.663	6.203	6.902

Bellota Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.653	0.655	0.658	0.659	0.659	0.658	0.658	0.656
10 min	0.651	0.657	0.658	0.658	0.660	0.658	0.657	0.657
15 min	0.651	0.656	0.659	0.659	0.660	0.659	0.659	0.656
30 min	0.652	0.657	0.659	0.660	0.659	0.659	0.658	0.657
60 min	0.650	0.657	0.659	0.660	0.660	0.659	0.659	0.657
3 hour	0.649	0.654	0.657	0.659	0.659	0.660	0.660	0.660
6 hour	0.648	0.653	0.656	0.658	0.659	0.660	0.661	0.662
12 hour	0.647	0.654	0.657	0.660	0.662	0.663	0.664	0.665
24 hour	0.647	0.654	0.658	0.661	0.662	0.663	0.664	0.665
48 hour	0.645	0.651	0.655	0.657	0.658	0.659	0.660	0.661
72 hour	0.645	0.650	0.652	0.654	0.655	0.656	0.657	0.658
96 hour	0.643	0.647	0.649	0.651	0.652	0.653	0.654	0.654

Bellota Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.083	0.114	0.139	0.171	0.196	0.220	0.245	0.278
10 min	0.118	0.164	0.199	0.245	0.282	0.316	0.351	0.399
15 min	0.143	0.197	0.241	0.297	0.341	0.383	0.426	0.482
30 min	0.200	0.275	0.335	0.414	0.473	0.532	0.592	0.671
60 min	0.270	0.373	0.454	0.561	0.642	0.721	0.803	0.909
3 hour	0.431	0.572	0.689	0.851	0.978	1.111	1.249	1.441
6 hour	0.576	0.759	0.914	1.128	1.297	1.474	1.662	1.925
12 hour	0.769	1.037	1.255	1.556	1.790	2.028	2.276	2.614
24 hour	1.047	1.432	1.742	2.155	2.464	2.774	3.090	3.511
48 hour	1.293	1.735	2.087	2.543	2.881	3.216	3.551	3.991
72 hour	1.474	1.953	2.325	2.811	3.170	3.522	3.874	4.330
96 hour	1.587	2.081	2.467	2.969	3.337	3.698	4.057	4.514

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FRM

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.127	0.174	0.211	0.260	0.298	0.335	0.373	0.424
10 min	0.182	0.249	0.302	0.373	0.427	0.480	0.534	0.607
15 min	0.220	0.301	0.365	0.451	0.516	0.581	0.646	0.734
30 min	0.306	0.418	0.508	0.627	0.717	0.808	0.899	1.021
60 min	0.415	0.567	0.689	0.850	0.972	1.094	1.218	1.384
3 hour	0.664	0.874	1.049	1.292	1.484	1.683	1.892	2.184
6 hour	0.889	1.163	1.393	1.714	1.968	2.234	2.515	2.908
12 hour	1.188	1.585	1.910	2.357	2.704	3.059	3.428	3.931
24 hour	1.619	2.190	2.648	3.260	3.722	4.184	4.654	5.280
48 hour	2.005	2.665	3.186	3.870	4.378	4.880	5.381	6.038
72 hour	2.286	3.004	3.566	4.298	4.839	5.369	5.896	6.581
96 hour	2.468	3.217	3.802	4.561	5.118	5.663	6.203	6.902

Above New Hogan Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.544	0.550	0.555	0.558	0.558	0.556	0.556	0.554
10 min	0.543	0.553	0.555	0.557	0.559	0.557	0.555	0.555
15 min	0.545	0.553	0.557	0.558	0.559	0.557	0.557	0.554
30 min	0.545	0.553	0.557	0.559	0.559	0.558	0.557	0.555
60 min	0.542	0.553	0.556	0.558	0.558	0.557	0.557	0.554
3 hour	0.534	0.543	0.548	0.551	0.553	0.554	0.555	0.555
6 hour	0.526	0.535	0.541	0.545	0.548	0.550	0.552	0.554
12 hour	0.519	0.531	0.538	0.544	0.546	0.549	0.551	0.552
24 hour	0.512	0.525	0.532	0.538	0.540	0.542	0.544	0.545
48 hour	0.501	0.512	0.518	0.522	0.525	0.526	0.527	0.529
72 hour	0.496	0.504	0.509	0.513	0.515	0.516	0.517	0.518
96 hour	0.490	0.498	0.501	0.504	0.506	0.508	0.509	0.510

Above New Hogan Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.069	0.096	0.117	0.145	0.166	0.186	0.207	0.235
10 min	0.099	0.138	0.168	0.208	0.239	0.267	0.296	0.337
15 min	0.120	0.166	0.203	0.252	0.288	0.324	0.360	0.407
30 min	0.167	0.231	0.283	0.350	0.401	0.451	0.501	0.567
60 min	0.225	0.314	0.383	0.474	0.542	0.609	0.678	0.767
3 hour	0.355	0.475	0.575	0.712	0.821	0.932	1.050	1.212
6 hour	0.468	0.622	0.754	0.934	1.078	1.229	1.388	1.611
12 hour	0.617	0.842	1.028	1.282	1.476	1.679	1.889	2.170
24 hour	0.829	1.150	1.409	1.754	2.010	2.268	2.532	2.878
48 hour	1.005	1.364	1.650	2.020	2.298	2.567	2.836	3.194
72 hour	1.134	1.514	1.815	2.205	2.492	2.770	3.048	3.409
96 hour	1.209	1.602	1.905	2.299	2.590	2.877	3.157	3.520

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FRM

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.127	0.174	0.211	0.260	0.298	0.335	0.373	0.424
10 min	0.182	0.249	0.302	0.373	0.427	0.480	0.534	0.607
15 min	0.220	0.301	0.365	0.451	0.516	0.581	0.646	0.734
30 min	0.306	0.418	0.508	0.627	0.717	0.808	0.899	1.021
60 min	0.415	0.567	0.689	0.850	0.972	1.094	1.218	1.384
3 hour	0.664	0.874	1.049	1.292	1.484	1.683	1.892	2.184
6 hour	0.889	1.163	1.393	1.714	1.968	2.234	2.515	2.908
12 hour	1.188	1.585	1.910	2.357	2.704	3.059	3.428	3.931
24 hour	1.619	2.190	2.648	3.260	3.722	4.184	4.654	5.280
48 hour	2.005	2.665	3.186	3.870	4.378	4.880	5.381	6.038
72 hour	2.286	3.004	3.566	4.298	4.839	5.369	5.896	6.581
96 hour	2.468	3.217	3.802	4.561	5.118	5.663	6.203	6.902

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.599	0.603	0.607	0.609	0.609	0.607	0.607	0.605
10 min	0.597	0.605	0.607	0.608	0.610	0.608	0.606	0.606
15 min	0.598	0.605	0.608	0.609	0.610	0.608	0.608	0.605
30 min	0.599	0.605	0.608	0.610	0.609	0.609	0.608	0.606
60 min	0.596	0.605	0.608	0.609	0.609	0.608	0.608	0.606
3 hour	0.592	0.599	0.603	0.605	0.606	0.607	0.608	0.608
6 hour	0.587	0.594	0.599	0.602	0.604	0.605	0.607	0.608
12 hour	0.583	0.593	0.598	0.602	0.604	0.606	0.608	0.609
24 hour	0.580	0.590	0.595	0.600	0.601	0.603	0.604	0.605
48 hour	0.573	0.582	0.587	0.590	0.592	0.593	0.594	0.595
72 hour	0.571	0.577	0.581	0.584	0.585	0.586	0.587	0.588
96 hour	0.567	0.573	0.575	0.578	0.579	0.581	0.582	0.582

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.076	0.105	0.128	0.158	0.181	0.203	0.226	0.257
10 min	0.109	0.151	0.183	0.227	0.260	0.292	0.324	0.368
15 min	0.132	0.182	0.222	0.275	0.315	0.353	0.393	0.444
30 min	0.183	0.253	0.309	0.382	0.437	0.492	0.547	0.619
60 min	0.247	0.343	0.419	0.518	0.592	0.665	0.741	0.839
3 hour	0.393	0.524	0.633	0.782	0.899	1.022	1.150	1.328
6 hour	0.522	0.691	0.834	1.032	1.189	1.352	1.527	1.768
12 hour	0.693	0.940	1.142	1.419	1.633	1.854	2.084	2.394
24 hour	0.939	1.292	1.576	1.956	2.237	2.523	2.811	3.194
48 hour	1.149	1.551	1.870	2.283	2.592	2.894	3.196	3.593
72 hour	1.305	1.733	2.072	2.510	2.831	3.146	3.461	3.870
96 hour	1.399	1.843	2.186	2.636	2.963	3.290	3.610	4.017

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: RBR

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.140	0.179	0.212	0.259	0.295	0.333	0.374	0.431
10 min	0.201	0.257	0.305	0.371	0.423	0.478	0.536	0.618
15 min	0.243	0.311	0.368	0.448	0.512	0.578	0.648	0.747
30 min	0.337	0.433	0.512	0.623	0.711	0.803	0.901	1.039
60 min	0.456	0.585	0.692	0.842	0.962	1.086	1.218	1.404
3 hour	0.751	0.929	1.080	1.294	1.467	1.649	1.846	2.125
6 hour	1.029	1.260	1.454	1.728	1.948	2.178	2.424	2.773
12 hour	1.377	1.692	1.952	2.312	2.593	2.884	3.187	3.606
24 hour	1.882	2.331	2.695	3.188	3.565	3.947	4.338	4.867
48 hour	2.368	2.936	3.388	3.988	4.438	4.886	5.338	5.935
72 hour	2.718	3.372	3.888	4.565	5.068	5.564	6.061	6.712
96 hour	2.997	3.722	4.290	5.030	5.576	6.112	6.645	7.339

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.717	0.715	0.715	0.715	0.716	0.715	0.715	0.716
10 min	0.716	0.715	0.714	0.715	0.714	0.715	0.716	0.717
15 min	0.715	0.715	0.716	0.715	0.715	0.716	0.716	0.716
30 min	0.718	0.715	0.715	0.716	0.715	0.716	0.716	0.717
60 min	0.716	0.714	0.714	0.714	0.714	0.715	0.715	0.716
3 hour	0.718	0.718	0.718	0.717	0.717	0.717	0.717	0.717
6 hour	0.719	0.719	0.719	0.719	0.719	0.718	0.718	0.717
12 hour	0.720	0.719	0.719	0.719	0.719	0.718	0.718	0.718
24 hour	0.719	0.719	0.718	0.718	0.718	0.717	0.717	0.717
48 hour	0.720	0.719	0.719	0.719	0.718	0.718	0.718	0.717
72 hour	0.720	0.720	0.720	0.719	0.719	0.719	0.718	0.718
96 hour	0.720	0.720	0.720	0.720	0.720	0.719	0.719	0.718

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.100	0.128	0.152	0.185	0.211	0.238	0.267	0.309
10 min	0.144	0.184	0.218	0.265	0.302	0.342	0.384	0.443
15 min	0.174	0.222	0.263	0.320	0.366	0.414	0.464	0.535
30 min	0.242	0.310	0.366	0.446	0.508	0.575	0.645	0.745
60 min	0.326	0.418	0.494	0.601	0.687	0.776	0.871	1.005
3 hour	0.539	0.667	0.775	0.928	1.052	1.182	1.324	1.524
6 hour	0.740	0.906	1.045	1.242	1.401	1.564	1.740	1.988
12 hour	0.991	1.217	1.403	1.662	1.864	2.071	2.288	2.589
24 hour	1.353	1.676	1.935	2.289	2.560	2.830	3.110	3.490
48 hour	1.705	2.111	2.436	2.867	3.186	3.508	3.833	4.255
72 hour	1.957	2.428	2.799	3.282	3.644	4.001	4.352	4.819
96 hour	2.158	2.680	3.089	3.622	4.015	4.395	4.778	5.269

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: RBR

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.140	0.179	0.212	0.259	0.295	0.333	0.374	0.431
10 min	0.201	0.257	0.305	0.371	0.423	0.478	0.536	0.618
15 min	0.243	0.311	0.368	0.448	0.512	0.578	0.648	0.747
30 min	0.337	0.433	0.512	0.623	0.711	0.803	0.901	1.039
60 min	0.456	0.585	0.692	0.842	0.962	1.086	1.218	1.404
3 hour	0.751	0.929	1.080	1.294	1.467	1.649	1.846	2.125
6 hour	1.029	1.260	1.454	1.728	1.948	2.178	2.424	2.773
12 hour	1.377	1.692	1.952	2.312	2.593	2.884	3.187	3.606
24 hour	1.882	2.331	2.695	3.188	3.565	3.947	4.338	4.867
48 hour	2.368	2.936	3.388	3.988	4.438	4.886	5.338	5.935
72 hour	2.718	3.372	3.888	4.565	5.068	5.564	6.061	6.712
96 hour	2.997	3.722	4.290	5.030	5.576	6.112	6.645	7.339

Bellota Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.732	0.733	0.731	0.732	0.733	0.731	0.732	0.731
10 min	0.734	0.734	0.731	0.732	0.731	0.732	0.732	0.732
15 min	0.732	0.733	0.733	0.733	0.732	0.732	0.732	0.732
30 min	0.733	0.732	0.732	0.732	0.732	0.731	0.731	0.731
60 min	0.734	0.732	0.733	0.732	0.732	0.733	0.732	0.732
3 hour	0.734	0.733	0.733	0.732	0.732	0.732	0.731	0.731
6 hour	0.734	0.733	0.733	0.732	0.732	0.732	0.731	0.731
12 hour	0.734	0.733	0.733	0.732	0.732	0.732	0.732	0.731
24 hour	0.733	0.733	0.733	0.732	0.732	0.732	0.732	0.732
48 hour	0.734	0.733	0.733	0.733	0.733	0.733	0.733	0.733
72 hour	0.734	0.733	0.733	0.733	0.733	0.733	0.733	0.733
96 hour	0.734	0.733	0.733	0.733	0.733	0.733	0.733	0.733

Bellota Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.102	0.131	0.155	0.190	0.216	0.243	0.274	0.315
10 min	0.148	0.189	0.223	0.272	0.309	0.350	0.392	0.452
15 min	0.178	0.228	0.270	0.328	0.375	0.423	0.474	0.547
30 min	0.247	0.317	0.375	0.456	0.520	0.587	0.659	0.760
60 min	0.335	0.428	0.507	0.616	0.704	0.796	0.892	1.028
3 hour	0.551	0.681	0.792	0.947	1.074	1.207	1.349	1.553
6 hour	0.755	0.924	1.066	1.265	1.426	1.594	1.772	2.027
12 hour	1.011	1.240	1.431	1.692	1.898	2.111	2.333	2.636
24 hour	1.380	1.709	1.975	2.334	2.610	2.889	3.175	3.563
48 hour	1.738	2.152	2.483	2.923	3.253	3.581	3.913	4.350
72 hour	1.995	2.472	2.850	3.346	3.715	4.078	4.443	4.920
96 hour	2.200	2.728	3.145	3.687	4.087	4.480	4.871	5.379

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: RBR

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.140	0.179	0.212	0.259	0.295	0.333	0.374	0.431
10 min	0.201	0.257	0.305	0.371	0.423	0.478	0.536	0.618
15 min	0.243	0.311	0.368	0.448	0.512	0.578	0.648	0.747
30 min	0.337	0.433	0.512	0.623	0.711	0.803	0.901	1.039
60 min	0.456	0.585	0.692	0.842	0.962	1.086	1.218	1.404
3 hour	0.751	0.929	1.080	1.294	1.467	1.649	1.846	2.125
6 hour	1.029	1.260	1.454	1.728	1.948	2.178	2.424	2.773
12 hour	1.377	1.692	1.952	2.312	2.593	2.884	3.187	3.606
24 hour	1.882	2.331	2.695	3.188	3.565	3.947	4.338	4.867
48 hour	2.368	2.936	3.388	3.988	4.438	4.886	5.338	5.935
72 hour	2.718	3.372	3.888	4.565	5.068	5.564	6.061	6.712
96 hour	2.997	3.722	4.290	5.030	5.576	6.112	6.645	7.339

Above New Hogan Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.600	0.603	0.601	0.604	0.604	0.602	0.602	0.603
10 min	0.602	0.603	0.602	0.603	0.603	0.603	0.603	0.603
15 min	0.601	0.603	0.604	0.604	0.603	0.603	0.603	0.603
30 min	0.603	0.602	0.603	0.603	0.604	0.603	0.603	0.603
60 min	0.602	0.601	0.603	0.603	0.603	0.603	0.603	0.602
3 hour	0.596	0.597	0.598	0.598	0.598	0.598	0.598	0.599
6 hour	0.592	0.592	0.592	0.593	0.593	0.594	0.594	0.594
12 hour	0.586	0.586	0.587	0.587	0.587	0.588	0.588	0.588
24 hour	0.580	0.580	0.581	0.581	0.581	0.581	0.581	0.581
48 hour	0.574	0.573	0.573	0.573	0.573	0.573	0.573	0.572
72 hour	0.570	0.569	0.568	0.568	0.568	0.568	0.568	0.568
96 hour	0.568	0.567	0.566	0.566	0.566	0.566	0.566	0.566

Above New Hogan Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.084	0.108	0.127	0.156	0.178	0.200	0.225	0.260
10 min	0.121	0.155	0.184	0.224	0.255	0.288	0.323	0.373
15 min	0.146	0.188	0.222	0.271	0.309	0.349	0.391	0.450
30 min	0.203	0.261	0.309	0.376	0.429	0.484	0.543	0.627
60 min	0.275	0.352	0.417	0.508	0.580	0.655	0.734	0.845
3 hour	0.448	0.555	0.646	0.774	0.877	0.986	1.104	1.273
6 hour	0.609	0.746	0.861	1.025	1.155	1.294	1.440	1.647
12 hour	0.807	0.992	1.146	1.357	1.522	1.696	1.874	2.120
24 hour	1.092	1.352	1.566	1.852	2.071	2.293	2.520	2.828
48 hour	1.359	1.682	1.941	2.285	2.543	2.800	3.059	3.395
72 hour	1.549	1.919	2.208	2.593	2.879	3.160	3.443	3.812
96 hour	1.702	2.110	2.428	2.847	3.156	3.459	3.761	4.154

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: RBR

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.140	0.179	0.212	0.259	0.295	0.333	0.374	0.431
10 min	0.201	0.257	0.305	0.371	0.423	0.478	0.536	0.618
15 min	0.243	0.311	0.368	0.448	0.512	0.578	0.648	0.747
30 min	0.337	0.433	0.512	0.623	0.711	0.803	0.901	1.039
60 min	0.456	0.585	0.692	0.842	0.962	1.086	1.218	1.404
3 hour	0.751	0.929	1.080	1.294	1.467	1.649	1.846	2.125
6 hour	1.029	1.260	1.454	1.728	1.948	2.178	2.424	2.773
12 hour	1.377	1.692	1.952	2.312	2.593	2.884	3.187	3.606
24 hour	1.882	2.331	2.695	3.188	3.565	3.947	4.338	4.867
48 hour	2.368	2.936	3.388	3.988	4.438	4.886	5.338	5.935
72 hour	2.718	3.372	3.888	4.565	5.068	5.564	6.061	6.712
96 hour	2.997	3.722	4.290	5.030	5.576	6.112	6.645	7.339

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.666	0.668	0.666	0.668	0.669	0.667	0.667	0.667
10 min	0.668	0.669	0.667	0.668	0.667	0.668	0.668	0.668
15 min	0.667	0.668	0.669	0.669	0.668	0.668	0.668	0.668
30 min	0.668	0.667	0.668	0.668	0.668	0.667	0.667	0.667
60 min	0.668	0.667	0.668	0.668	0.668	0.668	0.668	0.667
3 hour	0.665	0.665	0.666	0.665	0.665	0.665	0.665	0.665
6 hour	0.663	0.663	0.663	0.663	0.663	0.663	0.663	0.663
12 hour	0.660	0.660	0.660	0.660	0.660	0.660	0.660	0.660
24 hour	0.657	0.657	0.657	0.657	0.657	0.657	0.657	0.657
48 hour	0.654	0.653	0.653	0.653	0.653	0.653	0.653	0.653
72 hour	0.652	0.651	0.651	0.651	0.651	0.651	0.651	0.651
96 hour	0.651	0.650	0.650	0.650	0.650	0.650	0.650	0.650

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.093	0.120	0.141	0.173	0.197	0.222	0.249	0.287
10 min	0.134	0.172	0.203	0.248	0.282	0.319	0.358	0.413
15 min	0.162	0.208	0.246	0.300	0.342	0.386	0.433	0.499
30 min	0.225	0.289	0.342	0.416	0.475	0.536	0.601	0.693
60 min	0.305	0.390	0.462	0.562	0.643	0.725	0.814	0.936
3 hour	0.499	0.618	0.719	0.861	0.976	1.097	1.228	1.413
6 hour	0.682	0.835	0.964	1.146	1.292	1.444	1.607	1.838
12 hour	0.909	1.117	1.288	1.526	1.711	1.903	2.103	2.380
24 hour	1.236	1.531	1.771	2.095	2.342	2.593	2.850	3.198
48 hour	1.549	1.917	2.212	2.604	2.898	3.191	3.486	3.876
72 hour	1.772	2.195	2.531	2.972	3.299	3.622	3.946	4.370
96 hour	1.951	2.419	2.789	3.270	3.624	3.973	4.319	4.770

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: MDZ

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.148	0.191	0.227	0.277	0.318	0.360	0.405	0.468
10 min	0.212	0.274	0.325	0.398	0.455	0.516	0.580	0.671
15 min	0.257	0.331	0.393	0.481	0.551	0.624	0.702	0.812
30 min	0.359	0.463	0.550	0.672	0.769	0.872	0.981	1.134
60 min	0.479	0.618	0.734	0.897	1.027	1.164	1.309	1.514
3 hour	0.783	0.978	1.144	1.383	1.577	1.784	2.007	2.327
6 hour	1.073	1.328	1.544	1.851	2.099	2.360	2.640	3.037
12 hour	1.444	1.792	2.081	2.481	2.795	3.119	3.458	3.927
24 hour	1.980	2.472	2.870	3.406	3.815	4.229	4.652	5.222
48 hour	2.472	3.087	3.574	4.216	4.695	5.171	5.648	6.277
72 hour	2.848	3.558	4.114	4.837	5.371	5.895	6.416	7.095
96 hour	3.154	3.941	4.553	5.343	5.920	6.483	7.039	7.756

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.634	0.632	0.633	0.632	0.632	0.633	0.632	0.634
10 min	0.633	0.630	0.633	0.632	0.632	0.632	0.633	0.633
15 min	0.635	0.632	0.631	0.631	0.631	0.632	0.632	0.633
30 min	0.634	0.632	0.632	0.632	0.632	0.632	0.633	0.634
60 min	0.633	0.631	0.630	0.631	0.631	0.631	0.632	0.633
3 hour	0.634	0.633	0.632	0.632	0.633	0.633	0.633	0.633
6 hour	0.635	0.634	0.634	0.633	0.633	0.633	0.633	0.633
12 hour	0.635	0.634	0.634	0.633	0.632	0.632	0.632	0.632
24 hour	0.636	0.634	0.633	0.632	0.632	0.631	0.631	0.630
48 hour	0.636	0.634	0.634	0.633	0.632	0.632	0.632	0.631
72 hour	0.636	0.635	0.635	0.634	0.634	0.633	0.633	0.632
96 hour	0.637	0.636	0.636	0.635	0.635	0.635	0.634	0.634

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.094	0.121	0.144	0.175	0.201	0.228	0.256	0.297
10 min	0.134	0.173	0.206	0.252	0.288	0.326	0.367	0.425
15 min	0.163	0.209	0.248	0.304	0.348	0.394	0.444	0.514
30 min	0.228	0.293	0.348	0.425	0.486	0.551	0.621	0.719
60 min	0.303	0.390	0.462	0.566	0.648	0.734	0.827	0.958
3 hour	0.496	0.619	0.723	0.874	0.998	1.129	1.270	1.473
6 hour	0.681	0.842	0.979	1.172	1.329	1.494	1.671	1.922
12 hour	0.917	1.136	1.319	1.570	1.766	1.971	2.185	2.482
24 hour	1.259	1.567	1.817	2.153	2.411	2.668	2.935	3.290
48 hour	1.572	1.957	2.266	2.669	2.967	3.268	3.570	3.961
72 hour	1.811	2.259	2.612	3.067	3.405	3.732	4.061	4.484
96 hour	2.009	2.506	2.896	3.393	3.759	4.117	4.463	4.917

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: MDZ

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.148	0.191	0.227	0.277	0.318	0.360	0.405	0.468
10 min	0.212	0.274	0.325	0.398	0.455	0.516	0.580	0.671
15 min	0.257	0.331	0.393	0.481	0.551	0.624	0.702	0.812
30 min	0.359	0.463	0.550	0.672	0.769	0.872	0.981	1.134
60 min	0.479	0.618	0.734	0.897	1.027	1.164	1.309	1.514
3 hour	0.783	0.978	1.144	1.383	1.577	1.784	2.007	2.327
6 hour	1.073	1.328	1.544	1.851	2.099	2.360	2.640	3.037
12 hour	1.444	1.792	2.081	2.481	2.795	3.119	3.458	3.927
24 hour	1.980	2.472	2.870	3.406	3.815	4.229	4.652	5.222
48 hour	2.472	3.087	3.574	4.216	4.695	5.171	5.648	6.277
72 hour	2.848	3.558	4.114	4.837	5.371	5.895	6.416	7.095
96 hour	3.154	3.941	4.553	5.343	5.920	6.483	7.039	7.756

Bellota Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
10 min	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
15 min	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
30 min	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
60 min	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
3 hour	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
6 hour	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
12 hour	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
24 hour	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
48 hour	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
72 hour	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850
96 hour	0.850	0.850	0.850	0.850	0.850	0.850	0.850	0.850

Bellota Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.126	0.162	0.193	0.235	0.270	0.306	0.344	0.398
10 min	0.180	0.233	0.276	0.338	0.387	0.439	0.493	0.570
15 min	0.218	0.281	0.334	0.409	0.468	0.530	0.597	0.690
30 min	0.305	0.394	0.468	0.571	0.654	0.741	0.834	0.964
60 min	0.407	0.525	0.624	0.762	0.873	0.989	1.113	1.287
3 hour	0.666	0.831	0.972	1.176	1.340	1.516	1.706	1.978
6 hour	0.912	1.129	1.312	1.573	1.784	2.006	2.244	2.581
12 hour	1.227	1.523	1.769	2.109	2.376	2.651	2.939	3.338
24 hour	1.683	2.101	2.440	2.895	3.243	3.595	3.954	4.439
48 hour	2.101	2.624	3.038	3.584	3.991	4.395	4.801	5.335
72 hour	2.421	3.024	3.497	4.111	4.565	5.011	5.454	6.031
96 hour	2.681	3.350	3.870	4.542	5.032	5.511	5.983	6.593

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: MDZ

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.148	0.191	0.227	0.277	0.318	0.360	0.405	0.468
10 min	0.212	0.274	0.325	0.398	0.455	0.516	0.580	0.671
15 min	0.257	0.331	0.393	0.481	0.551	0.624	0.702	0.812
30 min	0.359	0.463	0.550	0.672	0.769	0.872	0.981	1.134
60 min	0.479	0.618	0.734	0.897	1.027	1.164	1.309	1.514
3 hour	0.783	0.978	1.144	1.383	1.577	1.784	2.007	2.327
6 hour	1.073	1.328	1.544	1.851	2.099	2.360	2.640	3.037
12 hour	1.444	1.792	2.081	2.481	2.795	3.119	3.458	3.927
24 hour	1.980	2.472	2.870	3.406	3.815	4.229	4.652	5.222
48 hour	2.472	3.087	3.574	4.216	4.695	5.171	5.648	6.277
72 hour	2.848	3.558	4.114	4.837	5.371	5.895	6.416	7.095
96 hour	3.154	3.941	4.553	5.343	5.920	6.483	7.039	7.756

Above New Hogan Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.588	0.592	0.595	0.597	0.595	0.596	0.595	0.596
10 min	0.589	0.589	0.596	0.595	0.597	0.596	0.595	0.595
15 min	0.593	0.593	0.592	0.595	0.595	0.594	0.596	0.596
30 min	0.591	0.593	0.595	0.596	0.597	0.597	0.597	0.596
60 min	0.588	0.590	0.591	0.593	0.594	0.594	0.594	0.594
3 hour	0.578	0.582	0.583	0.585	0.586	0.587	0.589	0.590
6 hour	0.569	0.573	0.575	0.577	0.579	0.580	0.581	0.583
12 hour	0.561	0.564	0.566	0.568	0.569	0.570	0.571	0.572
24 hour	0.552	0.554	0.555	0.557	0.557	0.557	0.557	0.557
48 hour	0.539	0.540	0.540	0.541	0.541	0.541	0.541	0.540
72 hour	0.534	0.534	0.533	0.533	0.533	0.533	0.532	0.532
96 hour	0.532	0.531	0.530	0.530	0.530	0.529	0.529	0.529

Above New Hogan Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.087	0.113	0.135	0.165	0.189	0.215	0.241	0.279
10 min	0.125	0.161	0.194	0.237	0.272	0.308	0.345	0.399
15 min	0.152	0.196	0.233	0.286	0.328	0.371	0.418	0.484
30 min	0.212	0.275	0.327	0.401	0.459	0.521	0.586	0.676
60 min	0.282	0.365	0.434	0.532	0.610	0.691	0.778	0.899
3 hour	0.453	0.569	0.667	0.809	0.924	1.047	1.182	1.373
6 hour	0.611	0.761	0.888	1.068	1.215	1.369	1.534	1.771
12 hour	0.810	1.011	1.178	1.409	1.590	1.778	1.975	2.246
24 hour	1.093	1.369	1.593	1.897	2.125	2.356	2.591	2.909
48 hour	1.332	1.667	1.930	2.281	2.540	2.798	3.056	3.390
72 hour	1.521	1.900	2.193	2.578	2.863	3.142	3.413	3.775
96 hour	1.678	2.093	2.413	2.832	3.138	3.430	3.724	4.103

CALAVERAS RIVER WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: MDZ

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.148	0.191	0.227	0.277	0.318	0.360	0.405	0.468
10 min	0.212	0.274	0.325	0.398	0.455	0.516	0.580	0.671
15 min	0.257	0.331	0.393	0.481	0.551	0.624	0.702	0.812
30 min	0.359	0.463	0.550	0.672	0.769	0.872	0.981	1.134
60 min	0.479	0.618	0.734	0.897	1.027	1.164	1.309	1.514
3 hour	0.783	0.978	1.144	1.383	1.577	1.784	2.007	2.327
6 hour	1.073	1.328	1.544	1.851	2.099	2.360	2.640	3.037
12 hour	1.444	1.792	2.081	2.481	2.795	3.119	3.458	3.927
24 hour	1.980	2.472	2.870	3.406	3.815	4.229	4.652	5.222
48 hour	2.472	3.087	3.574	4.216	4.695	5.171	5.648	6.277
72 hour	2.848	3.558	4.114	4.837	5.371	5.895	6.416	7.095
96 hour	3.154	3.941	4.553	5.343	5.920	6.483	7.039	7.756

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.719	0.721	0.723	0.724	0.723	0.723	0.723	0.723
10 min	0.720	0.720	0.723	0.723	0.724	0.723	0.723	0.723
15 min	0.722	0.722	0.721	0.723	0.723	0.722	0.723	0.723
30 min	0.721	0.722	0.723	0.723	0.724	0.724	0.724	0.723
60 min	0.719	0.720	0.721	0.722	0.722	0.722	0.722	0.722
3 hour	0.714	0.716	0.717	0.718	0.718	0.719	0.720	0.720
6 hour	0.710	0.712	0.713	0.714	0.715	0.715	0.716	0.717
12 hour	0.706	0.707	0.708	0.709	0.710	0.710	0.711	0.711
24 hour	0.701	0.702	0.703	0.704	0.704	0.704	0.704	0.704
48 hour	0.695	0.695	0.695	0.696	0.696	0.696	0.696	0.695
72 hour	0.692	0.692	0.692	0.692	0.692	0.692	0.691	0.691
96 hour	0.691	0.691	0.690	0.690	0.690	0.690	0.690	0.690

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.106	0.138	0.164	0.201	0.230	0.260	0.293	0.338
10 min	0.153	0.197	0.235	0.288	0.329	0.373	0.419	0.485
15 min	0.186	0.239	0.283	0.348	0.398	0.451	0.508	0.587
30 min	0.259	0.334	0.398	0.486	0.557	0.631	0.710	0.820
60 min	0.344	0.445	0.529	0.648	0.741	0.840	0.945	1.093
3 hour	0.559	0.700	0.820	0.993	1.132	1.283	1.445	1.675
6 hour	0.762	0.946	1.101	1.322	1.501	1.687	1.890	2.178
12 hour	1.019	1.267	1.473	1.759	1.984	2.214	2.459	2.792
24 hour	1.388	1.735	2.018	2.398	2.686	2.977	3.275	3.676
48 hour	1.718	2.145	2.484	2.934	3.268	3.599	3.931	4.363
72 hour	1.971	2.462	2.847	3.347	3.717	4.079	4.433	4.903
96 hour	2.179	2.723	3.142	3.687	4.085	4.473	4.857	5.352

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: NHG

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.168	0.218	0.261	0.323	0.372	0.425	0.482	0.565
10 min	0.241	0.313	0.375	0.462	0.534	0.609	0.691	0.810
15 min	0.292	0.379	0.453	0.559	0.645	0.737	0.836	0.979
30 min	0.406	0.527	0.631	0.778	0.898	1.026	1.164	1.364
60 min	0.542	0.703	0.841	1.038	1.198	1.368	1.552	1.817
3 hour	0.900	1.126	1.322	1.608	1.844	2.100	2.381	2.793
6 hour	1.267	1.567	1.824	2.194	2.495	2.818	3.168	3.673
12 hour	1.751	2.165	2.511	2.993	3.375	3.772	4.193	4.784
24 hour	2.467	3.062	3.544	4.196	4.694	5.200	5.721	6.428
48 hour	3.189	3.954	4.558	5.353	5.944	6.530	7.120	7.901
72 hour	3.740	4.636	5.334	6.238	6.902	7.552	8.198	9.042
96 hour	4.188	5.191	5.964	6.953	7.671	8.366	9.052	9.937

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.619	0.618	0.617	0.617	0.618	0.621	0.621	0.622
10 min	0.618	0.616	0.617	0.618	0.618	0.620	0.621	0.623
15 min	0.618	0.618	0.617	0.618	0.619	0.620	0.621	0.623
30 min	0.619	0.618	0.617	0.619	0.619	0.620	0.621	0.623
60 min	0.617	0.616	0.616	0.617	0.618	0.619	0.620	0.622
3 hour	0.620	0.619	0.619	0.619	0.619	0.620	0.621	0.622
6 hour	0.622	0.622	0.621	0.621	0.621	0.622	0.622	0.622
12 hour	0.625	0.623	0.623	0.622	0.622	0.622	0.622	0.622
24 hour	0.627	0.625	0.624	0.623	0.622	0.622	0.622	0.621
48 hour	0.629	0.627	0.627	0.626	0.625	0.625	0.624	0.624
72 hour	0.630	0.629	0.629	0.628	0.627	0.627	0.626	0.625
96 hour	0.632	0.631	0.630	0.629	0.629	0.628	0.627	0.627

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.104	0.135	0.161	0.199	0.230	0.264	0.299	0.351
10 min	0.149	0.193	0.231	0.286	0.330	0.378	0.429	0.505
15 min	0.180	0.234	0.280	0.345	0.399	0.457	0.519	0.610
30 min	0.251	0.326	0.389	0.482	0.556	0.636	0.723	0.850
60 min	0.334	0.433	0.518	0.640	0.740	0.847	0.962	1.130
3 hour	0.558	0.697	0.818	0.995	1.141	1.302	1.479	1.737
6 hour	0.788	0.975	1.133	1.362	1.549	1.753	1.970	2.285
12 hour	1.094	1.349	1.564	1.862	2.099	2.346	2.608	2.976
24 hour	1.547	1.914	2.211	2.614	2.920	3.234	3.558	3.992
48 hour	2.006	2.479	2.858	3.351	3.715	4.081	4.443	4.930
72 hour	2.356	2.916	3.355	3.917	4.328	4.735	5.132	5.651
96 hour	2.647	3.276	3.757	4.373	4.825	5.254	5.676	6.230

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: NHG

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.168	0.218	0.261	0.323	0.372	0.425	0.482	0.565
10 min	0.241	0.313	0.375	0.462	0.534	0.609	0.691	0.810
15 min	0.292	0.379	0.453	0.559	0.645	0.737	0.836	0.979
30 min	0.406	0.527	0.631	0.778	0.898	1.026	1.164	1.364
60 min	0.542	0.703	0.841	1.038	1.198	1.368	1.552	1.817
3 hour	0.900	1.126	1.322	1.608	1.844	2.100	2.381	2.793
6 hour	1.267	1.567	1.824	2.194	2.495	2.818	3.168	3.673
12 hour	1.751	2.165	2.511	2.993	3.375	3.772	4.193	4.784
24 hour	2.467	3.062	3.544	4.196	4.694	5.200	5.721	6.428
48 hour	3.189	3.954	4.558	5.353	5.944	6.530	7.120	7.901
72 hour	3.740	4.636	5.334	6.238	6.902	7.552	8.198	9.042
96 hour	4.188	5.191	5.964	6.953	7.671	8.366	9.052	9.937

Bellota Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.618	0.619	0.617	0.617	0.617	0.622	0.622	0.623
10 min	0.619	0.616	0.616	0.618	0.618	0.620	0.621	0.623
15 min	0.618	0.618	0.616	0.618	0.619	0.621	0.622	0.624
30 min	0.619	0.618	0.617	0.618	0.619	0.621	0.621	0.623
60 min	0.617	0.615	0.616	0.617	0.618	0.619	0.620	0.623
3 hour	0.620	0.619	0.618	0.618	0.618	0.619	0.620	0.622
6 hour	0.624	0.622	0.621	0.620	0.621	0.621	0.621	0.622
12 hour	0.627	0.624	0.623	0.621	0.621	0.620	0.620	0.620
24 hour	0.630	0.626	0.624	0.623	0.622	0.621	0.621	0.621
48 hour	0.633	0.630	0.629	0.627	0.626	0.626	0.625	0.625
72 hour	0.635	0.633	0.631	0.630	0.629	0.629	0.628	0.627
96 hour	0.637	0.635	0.634	0.633	0.632	0.631	0.630	0.629

Bellota Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.104	0.135	0.161	0.199	0.230	0.264	0.300	0.352
10 min	0.149	0.193	0.231	0.286	0.330	0.378	0.429	0.505
15 min	0.180	0.234	0.279	0.345	0.399	0.458	0.520	0.611
30 min	0.251	0.326	0.389	0.481	0.556	0.637	0.723	0.850
60 min	0.334	0.432	0.518	0.640	0.740	0.847	0.962	1.132
3 hour	0.558	0.697	0.817	0.994	1.140	1.300	1.476	1.737
6 hour	0.791	0.975	1.133	1.360	1.549	1.750	1.967	2.285
12 hour	1.098	1.351	1.564	1.859	2.096	2.339	2.600	2.966
24 hour	1.554	1.917	2.211	2.614	2.920	3.229	3.553	3.992
48 hour	2.019	2.491	2.867	3.356	3.721	4.088	4.450	4.938
72 hour	2.375	2.935	3.366	3.930	4.341	4.750	5.148	5.669
96 hour	2.668	3.296	3.781	4.401	4.848	5.279	5.703	6.250

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: NHG

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.168	0.218	0.261	0.323	0.372	0.425	0.482	0.565
10 min	0.241	0.313	0.375	0.462	0.534	0.609	0.691	0.810
15 min	0.292	0.379	0.453	0.559	0.645	0.737	0.836	0.979
30 min	0.406	0.527	0.631	0.778	0.898	1.026	1.164	1.364
60 min	0.542	0.703	0.841	1.038	1.198	1.368	1.552	1.817
3 hour	0.900	1.126	1.322	1.608	1.844	2.100	2.381	2.793
6 hour	1.267	1.567	1.824	2.194	2.495	2.818	3.168	3.673
12 hour	1.751	2.165	2.511	2.993	3.375	3.772	4.193	4.784
24 hour	2.467	3.062	3.544	4.196	4.694	5.200	5.721	6.428
48 hour	3.189	3.954	4.558	5.353	5.944	6.530	7.120	7.901
72 hour	3.740	4.636	5.334	6.238	6.902	7.552	8.198	9.042
96 hour	4.188	5.191	5.964	6.953	7.671	8.366	9.052	9.937

Above New Hogan Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.639	0.645	0.645	0.648	0.648	0.652	0.651	0.652
10 min	0.640	0.641	0.645	0.647	0.649	0.650	0.651	0.653
15 min	0.641	0.645	0.645	0.649	0.650	0.651	0.652	0.653
30 min	0.642	0.645	0.646	0.649	0.651	0.652	0.652	0.653
60 min	0.637	0.640	0.644	0.646	0.647	0.649	0.650	0.651
3 hour	0.631	0.635	0.636	0.639	0.640	0.643	0.644	0.647
6 hour	0.627	0.629	0.631	0.633	0.636	0.637	0.639	0.641
12 hour	0.622	0.624	0.626	0.627	0.629	0.630	0.631	0.632
24 hour	0.618	0.619	0.619	0.620	0.620	0.620	0.620	0.621
48 hour	0.612	0.611	0.611	0.610	0.610	0.610	0.609	0.608
72 hour	0.610	0.608	0.607	0.606	0.605	0.605	0.604	0.603
96 hour	0.610	0.608	0.606	0.605	0.604	0.603	0.602	0.601

Above New Hogan Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.107	0.141	0.168	0.209	0.241	0.277	0.314	0.368
10 min	0.154	0.201	0.242	0.299	0.347	0.396	0.450	0.529
15 min	0.187	0.244	0.292	0.363	0.419	0.480	0.545	0.639
30 min	0.261	0.340	0.408	0.505	0.585	0.669	0.759	0.891
60 min	0.345	0.450	0.542	0.671	0.775	0.888	1.009	1.183
3 hour	0.568	0.715	0.841	1.028	1.180	1.350	1.533	1.807
6 hour	0.794	0.986	1.151	1.389	1.587	1.795	2.024	2.354
12 hour	1.089	1.351	1.572	1.877	2.123	2.376	2.646	3.023
24 hour	1.525	1.895	2.194	2.602	2.910	3.224	3.547	3.992
48 hour	1.952	2.416	2.785	3.265	3.626	3.983	4.336	4.804
72 hour	2.281	2.819	3.238	3.780	4.176	4.569	4.952	5.452
96 hour	2.555	3.156	3.614	4.207	4.633	5.045	5.449	5.972

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: NHG

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.168	0.218	0.261	0.323	0.372	0.425	0.482	0.565
10 min	0.241	0.313	0.375	0.462	0.534	0.609	0.691	0.810
15 min	0.292	0.379	0.453	0.559	0.645	0.737	0.836	0.979
30 min	0.406	0.527	0.631	0.778	0.898	1.026	1.164	1.364
60 min	0.542	0.703	0.841	1.038	1.198	1.368	1.552	1.817
3 hour	0.900	1.126	1.322	1.608	1.844	2.100	2.381	2.793
6 hour	1.267	1.567	1.824	2.194	2.495	2.818	3.168	3.673
12 hour	1.751	2.165	2.511	2.993	3.375	3.772	4.193	4.784
24 hour	2.467	3.062	3.544	4.196	4.694	5.200	5.721	6.428
48 hour	3.189	3.954	4.558	5.353	5.944	6.530	7.120	7.901
72 hour	3.740	4.636	5.334	6.238	6.902	7.552	8.198	9.042
96 hour	4.188	5.191	5.964	6.953	7.671	8.366	9.052	9.937

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.629	0.632	0.631	0.633	0.633	0.637	0.637	0.638
10 min	0.630	0.629	0.631	0.633	0.634	0.635	0.636	0.638
15 min	0.630	0.632	0.631	0.634	0.635	0.636	0.637	0.639
30 min	0.631	0.632	0.632	0.634	0.635	0.637	0.637	0.638
60 min	0.627	0.628	0.630	0.632	0.633	0.634	0.635	0.637
3 hour	0.626	0.627	0.627	0.629	0.629	0.631	0.632	0.635
6 hour	0.626	0.626	0.626	0.627	0.629	0.629	0.630	0.632
12 hour	0.625	0.624	0.625	0.624	0.625	0.625	0.626	0.626
24 hour	0.624	0.623	0.622	0.622	0.621	0.621	0.621	0.621
48 hour	0.623	0.621	0.620	0.619	0.618	0.618	0.617	0.617
72 hour	0.623	0.621	0.619	0.618	0.617	0.617	0.616	0.615
96 hour	0.624	0.622	0.620	0.619	0.618	0.617	0.616	0.615

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.106	0.138	0.165	0.204	0.235	0.271	0.307	0.360
10 min	0.152	0.197	0.237	0.292	0.339	0.387	0.439	0.517
15 min	0.184	0.240	0.286	0.354	0.410	0.469	0.533	0.626
30 min	0.256	0.333	0.399	0.493	0.570	0.654	0.741	0.870
60 min	0.340	0.441	0.530	0.656	0.758	0.867	0.986	1.157
3 hour	0.563	0.706	0.829	1.011	1.160	1.325	1.505	1.774
6 hour	0.793	0.981	1.142	1.376	1.569	1.773	1.996	2.321
12 hour	1.094	1.351	1.569	1.868	2.109	2.358	2.625	2.995
24 hour	1.539	1.908	2.204	2.610	2.915	3.229	3.553	3.992
48 hour	1.987	2.455	2.826	3.314	3.673	4.036	4.393	4.875
72 hour	2.330	2.879	3.302	3.855	4.259	4.660	5.050	5.561
96 hour	2.613	3.229	3.698	4.304	4.741	5.162	5.576	6.111

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: UPPER CAL

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.176	0.221	0.260	0.314	0.357	0.403	0.451	0.521
10 min	0.252	0.317	0.372	0.450	0.512	0.577	0.647	0.747
15 min	0.305	0.384	0.450	0.544	0.619	0.698	0.782	0.904
30 min	0.420	0.529	0.620	0.749	0.853	0.962	1.078	1.246
60 min	0.576	0.725	0.851	1.028	1.169	1.319	1.479	1.708
3 hour	1.015	1.241	1.432	1.700	1.916	2.143	2.385	2.731
6 hour	1.487	1.801	2.062	2.428	2.718	3.020	3.340	3.788
12 hour	2.129	2.587	2.965	3.485	3.892	4.310	4.748	5.352
24 hour	3.103	3.816	4.396	5.183	5.787	6.401	7.035	7.896
48 hour	4.189	5.210	6.023	7.099	7.908	8.714	9.530	10.618
72 hour	4.980	6.240	7.228	8.519	9.476	10.420	11.367	12.613
96 hour	5.578	7.014	8.130	9.571	10.629	11.663	12.690	14.029

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.567	0.562	0.561	0.559	0.560	0.559	0.560	0.560
10 min	0.566	0.563	0.561	0.560	0.559	0.560	0.560	0.561
15 min	0.565	0.562	0.561	0.559	0.560	0.560	0.559	0.560
30 min	0.565	0.562	0.560	0.560	0.559	0.559	0.559	0.560
60 min	0.567	0.564	0.561	0.560	0.560	0.560	0.560	0.561
3 hour	0.573	0.570	0.569	0.568	0.567	0.566	0.565	0.564
6 hour	0.578	0.576	0.574	0.573	0.572	0.571	0.570	0.569
12 hour	0.582	0.580	0.578	0.577	0.576	0.575	0.575	0.574
24 hour	0.585	0.583	0.582	0.581	0.581	0.580	0.580	0.580
48 hour	0.589	0.589	0.588	0.588	0.587	0.587	0.587	0.587
72 hour	0.591	0.591	0.591	0.591	0.591	0.590	0.590	0.590
96 hour	0.592	0.592	0.592	0.592	0.592	0.592	0.592	0.592

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.100	0.124	0.146	0.176	0.200	0.225	0.253	0.292
10 min	0.143	0.178	0.209	0.252	0.286	0.323	0.362	0.419
15 min	0.172	0.216	0.252	0.304	0.347	0.391	0.437	0.506
30 min	0.237	0.297	0.347	0.419	0.477	0.538	0.603	0.698
60 min	0.327	0.409	0.477	0.576	0.655	0.739	0.828	0.958
3 hour	0.582	0.707	0.815	0.966	1.086	1.213	1.348	1.540
6 hour	0.859	1.037	1.184	1.391	1.555	1.724	1.904	2.155
12 hour	1.239	1.500	1.714	2.011	2.242	2.478	2.730	3.072
24 hour	1.815	2.225	2.558	3.011	3.362	3.713	4.080	4.580
48 hour	2.467	3.069	3.542	4.174	4.642	5.115	5.594	6.233
72 hour	2.943	3.688	4.272	5.035	5.600	6.148	6.707	7.442
96 hour	3.302	4.152	4.813	5.666	6.292	6.904	7.512	8.305

CALAVERAS RIVER WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: UPPER CAL

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.176	0.221	0.260	0.314	0.357	0.403	0.451	0.521
10 min	0.252	0.317	0.372	0.450	0.512	0.577	0.647	0.747
15 min	0.305	0.384	0.450	0.544	0.619	0.698	0.782	0.904
30 min	0.420	0.529	0.620	0.749	0.853	0.962	1.078	1.246
60 min	0.576	0.725	0.851	1.028	1.169	1.319	1.479	1.708
3 hour	1.015	1.241	1.432	1.700	1.916	2.143	2.385	2.731
6 hour	1.487	1.801	2.062	2.428	2.718	3.020	3.340	3.788
12 hour	2.129	2.587	2.965	3.485	3.892	4.310	4.748	5.352
24 hour	3.103	3.816	4.396	5.183	5.787	6.401	7.035	7.896
48 hour	4.189	5.210	6.023	7.099	7.908	8.714	9.530	10.618
72 hour	4.980	6.240	7.228	8.519	9.476	10.420	11.367	12.613
96 hour	5.578	7.014	8.130	9.571	10.629	11.663	12.690	14.029

Bellota Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.610	0.607	0.606	0.603	0.604	0.604	0.604	0.604
10 min	0.609	0.607	0.605	0.605	0.604	0.604	0.604	0.604
15 min	0.608	0.606	0.605	0.604	0.604	0.604	0.603	0.604
30 min	0.608	0.606	0.605	0.604	0.603	0.603	0.603	0.604
60 min	0.610	0.608	0.606	0.605	0.605	0.604	0.604	0.605
3 hour	0.615	0.612	0.611	0.610	0.609	0.608	0.608	0.607
6 hour	0.618	0.616	0.615	0.614	0.613	0.612	0.611	0.610
12 hour	0.621	0.619	0.618	0.617	0.616	0.615	0.615	0.614
24 hour	0.624	0.622	0.621	0.620	0.620	0.620	0.619	0.619
48 hour	0.627	0.626	0.626	0.626	0.625	0.625	0.625	0.625
72 hour	0.629	0.628	0.628	0.628	0.628	0.628	0.628	0.628
96 hour	0.630	0.629	0.629	0.629	0.629	0.629	0.629	0.629

Bellota Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.107	0.134	0.158	0.189	0.216	0.243	0.272	0.315
10 min	0.153	0.192	0.225	0.272	0.309	0.349	0.391	0.451
15 min	0.185	0.233	0.272	0.329	0.374	0.422	0.472	0.546
30 min	0.255	0.321	0.375	0.452	0.514	0.580	0.650	0.753
60 min	0.351	0.441	0.516	0.622	0.707	0.797	0.893	1.033
3 hour	0.624	0.759	0.875	1.037	1.167	1.303	1.450	1.658
6 hour	0.919	1.109	1.268	1.491	1.666	1.848	2.041	2.311
12 hour	1.322	1.601	1.832	2.150	2.397	2.651	2.920	3.286
24 hour	1.936	2.374	2.730	3.213	3.588	3.969	4.355	4.888
48 hour	2.627	3.261	3.770	4.444	4.943	5.446	5.956	6.636
72 hour	3.132	3.919	4.539	5.350	5.951	6.544	7.138	7.921
96 hour	3.514	4.412	5.114	6.020	6.686	7.336	7.982	8.824

CALAVERAS RIVER WATERSHED

NOAA14 Precipitation Frequency Depths

Rainfall Zone: UPPER CAL

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.176	0.221	0.260	0.314	0.357	0.403	0.451	0.521
10 min	0.252	0.317	0.372	0.450	0.512	0.577	0.647	0.747
15 min	0.305	0.384	0.450	0.544	0.619	0.698	0.782	0.904
30 min	0.420	0.529	0.620	0.749	0.853	0.962	1.078	1.246
60 min	0.576	0.725	0.851	1.028	1.169	1.319	1.479	1.708
3 hour	1.015	1.241	1.432	1.700	1.916	2.143	2.385	2.731
6 hour	1.487	1.801	2.062	2.428	2.718	3.020	3.340	3.788
12 hour	2.129	2.587	2.965	3.485	3.892	4.310	4.748	5.352
24 hour	3.103	3.816	4.396	5.183	5.787	6.401	7.035	7.896
48 hour	4.189	5.210	6.023	7.099	7.908	8.714	9.530	10.618
72 hour	4.980	6.240	7.228	8.519	9.476	10.420	11.367	12.613
96 hour	5.578	7.014	8.130	9.571	10.629	11.663	12.690	14.029

Above New Hogan Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
10 min	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
15 min	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
30 min	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
60 min	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
3 hour	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
6 hour	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
12 hour	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
24 hour	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
48 hour	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
72 hour	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764
96 hour	0.764	0.764	0.764	0.764	0.764	0.764	0.764	0.764

Above New Hogan Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.134	0.169	0.199	0.240	0.273	0.308	0.345	0.398
10 min	0.193	0.242	0.284	0.344	0.391	0.441	0.494	0.571
15 min	0.233	0.293	0.344	0.416	0.473	0.533	0.597	0.691
30 min	0.321	0.404	0.474	0.572	0.652	0.735	0.824	0.952
60 min	0.440	0.554	0.650	0.785	0.893	1.008	1.130	1.305
3 hour	0.775	0.948	1.094	1.299	1.464	1.637	1.822	2.086
6 hour	1.136	1.376	1.575	1.855	2.077	2.307	2.552	2.894
12 hour	1.627	1.976	2.265	2.663	2.973	3.293	3.627	4.089
24 hour	2.371	2.915	3.359	3.960	4.421	4.890	5.375	6.033
48 hour	3.200	3.980	4.602	5.424	6.042	6.657	7.281	8.112
72 hour	3.805	4.767	5.522	6.509	7.240	7.961	8.684	9.636
96 hour	4.262	5.359	6.211	7.312	8.121	8.911	9.695	10.718

CALAVERAS RIVER WATERSHED
NOAA14 Precipitation Frequency Depths
Rainfall Zone: UPPER CAL

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.176	0.221	0.260	0.314	0.357	0.403	0.451	0.521
10 min	0.252	0.317	0.372	0.450	0.512	0.577	0.647	0.747
15 min	0.305	0.384	0.450	0.544	0.619	0.698	0.782	0.904
30 min	0.420	0.529	0.620	0.749	0.853	0.962	1.078	1.246
60 min	0.576	0.725	0.851	1.028	1.169	1.319	1.479	1.708
3 hour	1.015	1.241	1.432	1.700	1.916	2.143	2.385	2.731
6 hour	1.487	1.801	2.062	2.428	2.718	3.020	3.340	3.788
12 hour	2.129	2.587	2.965	3.485	3.892	4.310	4.748	5.352
24 hour	3.103	3.816	4.396	5.183	5.787	6.401	7.035	7.896
48 hour	4.189	5.210	6.023	7.099	7.908	8.714	9.530	10.618
72 hour	4.980	6.240	7.228	8.519	9.476	10.420	11.367	12.613
96 hour	5.578	7.014	8.130	9.571	10.629	11.663	12.690	14.029

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.687	0.686	0.685	0.684	0.684	0.684	0.684	0.684
10 min	0.687	0.686	0.685	0.685	0.684	0.684	0.684	0.684
15 min	0.686	0.685	0.685	0.684	0.684	0.684	0.684	0.684
30 min	0.686	0.685	0.685	0.684	0.684	0.684	0.684	0.684
60 min	0.687	0.686	0.685	0.685	0.685	0.684	0.684	0.685
3 hour	0.690	0.688	0.688	0.687	0.687	0.686	0.686	0.686
6 hour	0.691	0.690	0.690	0.689	0.689	0.688	0.688	0.687
12 hour	0.693	0.692	0.691	0.691	0.690	0.690	0.690	0.689
24 hour	0.694	0.693	0.693	0.692	0.692	0.692	0.692	0.692
48 hour	0.696	0.695	0.695	0.695	0.695	0.695	0.695	0.695
72 hour	0.697	0.696	0.696	0.696	0.696	0.696	0.696	0.696
96 hour	0.697	0.697	0.697	0.697	0.697	0.697	0.697	0.697

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.121	0.152	0.178	0.215	0.244	0.276	0.308	0.356
10 min	0.173	0.217	0.255	0.308	0.350	0.395	0.443	0.511
15 min	0.209	0.263	0.308	0.372	0.423	0.477	0.535	0.618
30 min	0.288	0.362	0.425	0.512	0.583	0.658	0.737	0.852
60 min	0.396	0.497	0.583	0.704	0.801	0.902	1.012	1.170
3 hour	0.700	0.854	0.985	1.168	1.316	1.470	1.636	1.873
6 hour	1.028	1.243	1.423	1.673	1.873	2.078	2.298	2.602
12 hour	1.475	1.790	2.049	2.408	2.685	2.974	3.276	3.688
24 hour	2.153	2.644	3.046	3.587	4.005	4.429	4.868	5.464
48 hour	2.916	3.621	4.186	4.934	5.496	6.056	6.623	7.380
72 hour	3.471	4.343	5.031	5.929	6.595	7.252	7.911	8.779
96 hour	3.888	4.889	5.667	6.671	7.408	8.129	8.845	9.778

Attachment 5- E. ITR Comment Forms for Calaveras River HEC-HMS Modeling

Lower San Joaquin River Feasibility Study

HYDROLOGIC MODELING REVIEW – CALAVERAS RIVER WATERSHED

Task Order 6 – LSJRFS Work-In-Kind Hydrology

Reviewer:	Domenichelli & Associates
Review Date:	11-01-10
PBI Response Date:	11-03-10
DA Backcheck:	11-15-10

Memorandum Comments:

1. *Section 5.4.1 Subbasins – This would be a good place to talk about element HOLM and HOLM-PS. Element C80 from the HEC-1 model has been removed and replaced with HOLM and HOLM-PS.*

   PBI Response: An explanation for the re-naming of subbasin C80 has been added to *Section 5.4.1: Subbasins*.

   DA Backcheck: Accepted.

2. *In Attachment A the HEC-1 vs HEC-HMS comparison there is a model element RSRES and R60 but these elements are not found in Table 2. Summary of PBI Model routing elements or in the existing model. Add a paragraph explaining the removal of these elements or if the removal was unintentional add the elements to the model.*

   PBI Response: Three reach elements from the HEC-1 model (RSRES, R60, and RS4060) were initially combined into 1 reach (R4070) for the PBI Model. They have now been separated back into their original components for clarity purposes. See Table 2.

   DA Backcheck 1: R4070 is still in Table 2 from C40 to C70. RDV40, RSRES, and R60 are also in the model covering C40 to C70.

   DA Backcheck 2: The new length for RDV40, RSRES, and R60 is now 4000 feet more than the original length for R4070, which is correct?

   PBI Response to Backcheck 1: R4070 is a routing reach that represents the main channel of the Calaveras River from C40 to C70. RDV40, RSRES, and R60 are routing reaches that were added in to the model to represent overland flow from C40 to C70 for that portion of subbasin runoff that is prevented from entering the main channel of the Calaveras River due to levee barriers.

PBI Response to Backcheck 2: RDV40, RSRES, and R60 represent overland routing which takes a longer pathway than the main channel path of R4070.

Model Comments:

3. *In the model the impervious percentage of basin CG10 is listed as 0% impervious. From the figures in the memorandum it appears that the La Contenta community is within the basin. Consider using an impervious percentage of 2% or 5% for that basin.*

   PBI Response: Agreed. Subbasin CG10 is now assigned an impervious percentage of 5%.

   DA Backcheck: Accepted per attachment C.

Attachment 5- F. SPK Comment Forms for Calaveras River HEC-HMS Modeling

Corps of Engineers, Hydrology Section

Review of Calaveras HEC-1 to HEC-HMS model conversion and preliminary report.

22 November 2010 (Revised and transmitted 30 November 2010, sfh)

by Steven F. Holmstrom, P.E.

The Technical Memorandum for the Lower San Joaquin River Feasibility Study Calaveras HEC-HMS modeling DRAFT hydrology report has been reviewed and the following comments are provided.

18. In section 5.3 Design Storms, the fourth paragraph states that three storm centerings will be analyzed. A third storm centering will be required to compute the flow above the New Hogan Dam for the 8 design storm frequencies. Thank you for adding a third storm centering.

    PBI Response: No response necessary.

19. In Section 5.4.2 Reservoirs and Pumps, it must be made clear that the pumps discharge into the receiving channel above the highest stage to be expected so that there is independence between the exterior and interior areas. If that is not the case then a coincidence analysis must be performed to determine the modified interior pond stage-frequency curve considering the exterior-interior stage conditions. This is explained in EM1110-2-1413, Hydrologic Analysis of Interior Areas.

    PBI Response: Section 5.4.2 has been updated.

20. In figure 2, Calaveras River subbasins, the precipitation gage at Perry Ranch (PRY) should be shown, as it is mentioned in paragraph 5.5 model calibration. In addition, the stream gage at Duck Creek near Farmington that was mentioned in paragraph 5.2.1 SJAFCA HEC-1 model should be shown on the figure. A comparison of the 1/100 AEP results from the previous study and the current study will be interesting to see when the NOAA Atlas 14 document is published.

    PBI Response: Figure 2 now includes the mentioned gages.

    A comparison of the 1/100 AEP results from the 1998 HEC-1 study and the current study can be included once NOAA Atlas 14 precipitation is coded into the HEC-HMS model and production runs are completed.

21. The HMS model transmitted with this report failed at reservoir element “STPON”. Results from the report could not be compared with either the results from the 1998 study or the calibration done for the current study. The report must clarify the purpose and the state of the input parameters contained in the model supplied. It has been determined that the available storage for element “STPON” is marginal relative to the event simulated and events more rare. Additional storage should be coded into the model if available. Alternatively, an emergency flow path should be defined should the detention pond overflow.

    PBI Response: After discussions with the Corps, it was found that the transmitted model ran to completion when using HEC-HMS v3.4, but not when using the recently released HEC-HMS v3.5. The cause of this was determined to be an elevation-storage function for “STPON” that did not define the relationship for the upper limit of simulated storage conditions. To remedy this, the STPON function was inspected and extrapolated to handle a larger inflow event. All storage functions will be inspected once NOAA14 precipitation events are coded into the model to ensure they can handle the 500-year event (and beyond).

Attachment 6- A. Summary of Isolated Areas for French Camp Slough Subbasins

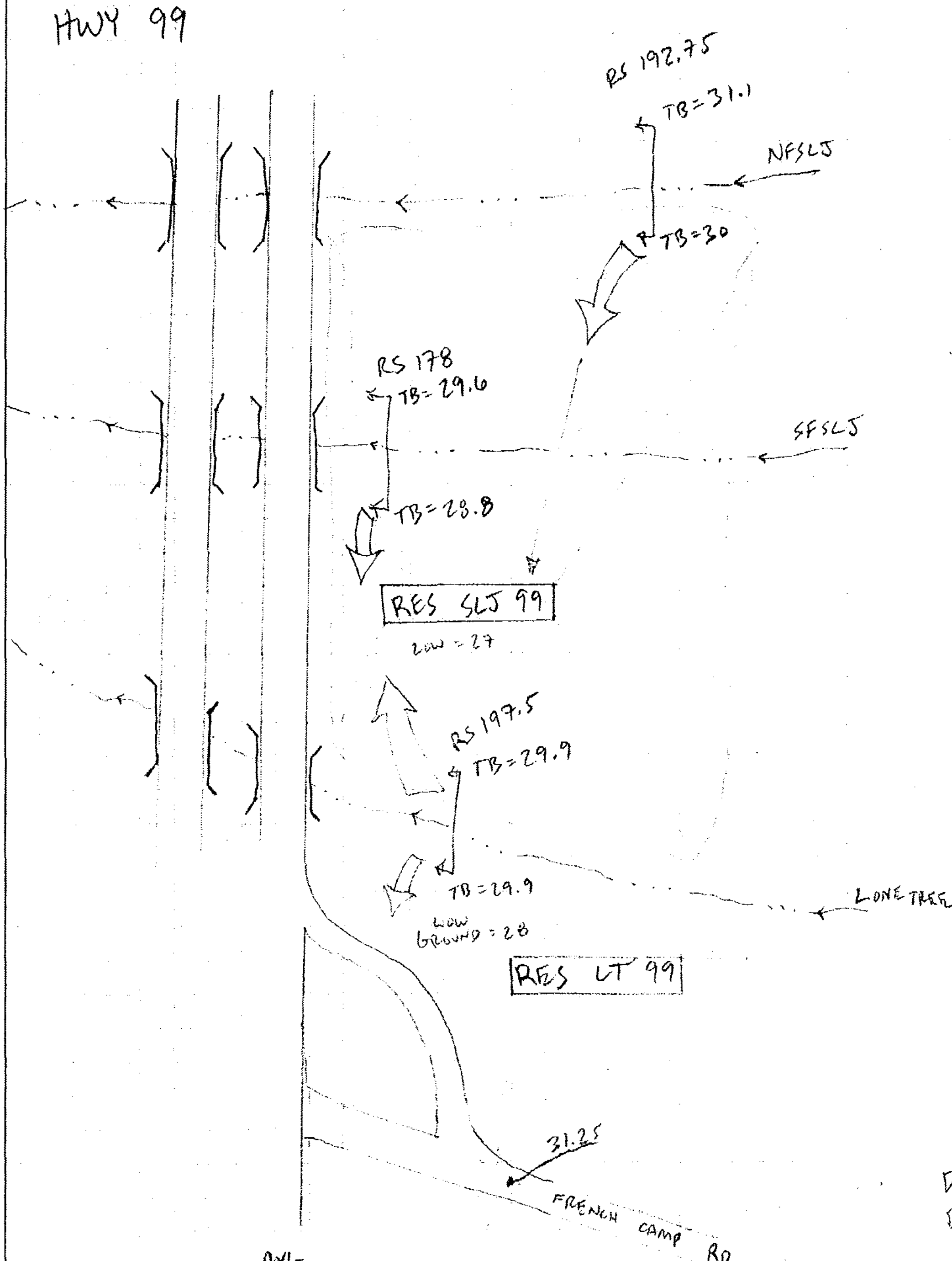
Subbasin	Measured Area	Percent of Area Estimated to be Isolated[a]	Area Used in Model
	[Sq. Mi.]	[%]	[sq. mi.]
LT A1	2.44	5%	2.32
LT A2	4.00	5%	3.80
LT A3	0.16	10%	0.15
LT B1	3.55	10%	3.20
LT B2	3.43	5%	3.26
LT B5	2.41	15%	2.05
LT C1	2.91	15%	2.47
LT C2	3.13	15%	2.66
LT C3	1.02	5%	0.97
LT C4a	1.87	10%	1.68
LT C4b	1.58	25%	1.19
LT D1	3.90	10%	3.51
LT D2	2.65	15%	2.25
TE A1	3.67	5%	3.49
TE B1	3.62	25%	2.72
TE B2	3.03	25%	2.27
TE B4	3.77	25%	2.83
TE C1	3.32	20%	2.66
TE D2	7.12	25%	5.34
TE D3	3.62	20%	2.90
TE F1	6.04	15%	5.13
TE F2	4.09	15%	3.47
TOTAL (Includes all 85 subbasins)	428.38	2.6%	417.35

[a] Percentages are based on field investigations conducted for the 2007 Tidewater Model

Attachment 6- B. Drawings and Hydraulic Calculations from the 2007 Tidewater Study

TIDEWATER

HWY 99



RS 192.75

Q	ELEV	DIV
1250	30.1	30
1530	30.6	200
1750	31.2	400
2000	31.8	550

RS 178

Q	ELEV	DIV
1750	29.0	10
2000	29.5	220
2300	30	450
2500	30.25	600

RS 197.5

Q	ELEV	DIV
750	29.6	0
1000	30.4	150
1250	31.0	350
1500	31.6	550
1750	32.1	700
2000	32.6	850

DIV R = 0.75 DIV
DIV L = 0.25 DIV

RES SLJ99

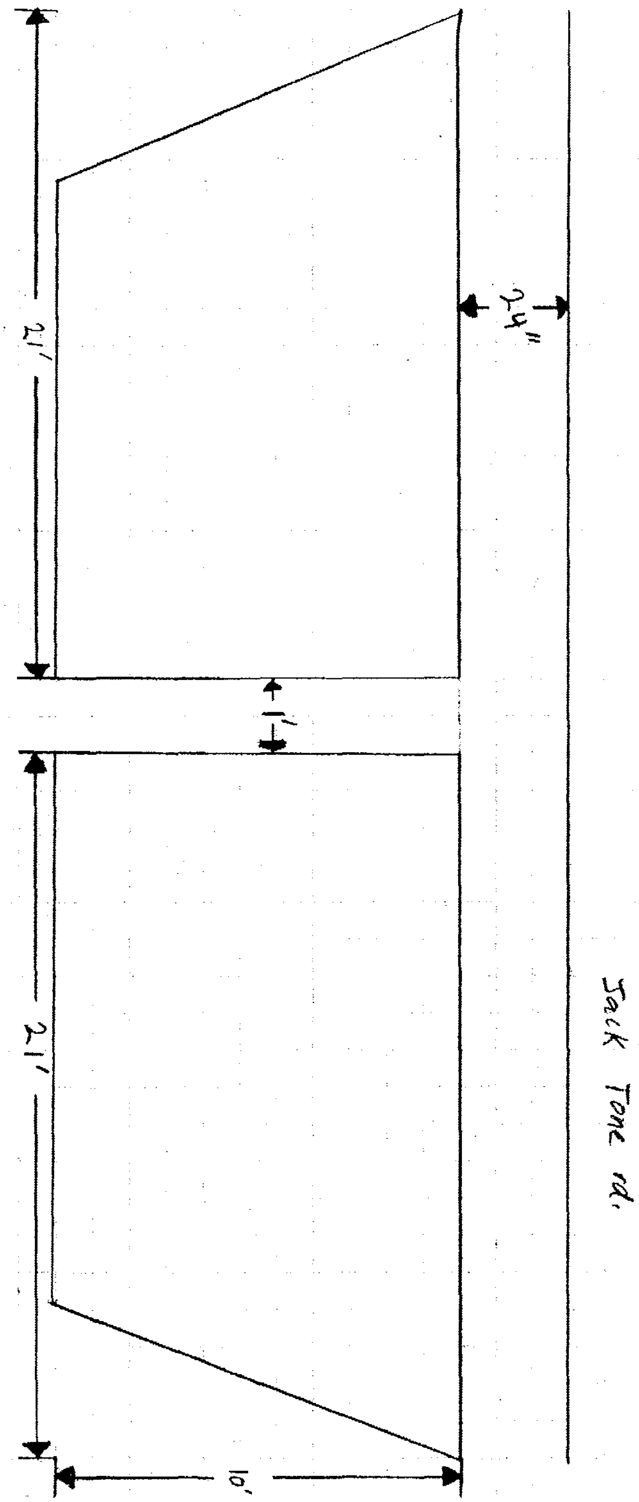
ELEV	AREA	AVG DEPTH	STORAGE	OUTFLOW
29	160	2	320	10
30	293	2.5	730	20
31	480	3	1440	35
32	1215	3.3	4000	200
33	—	4	—	—

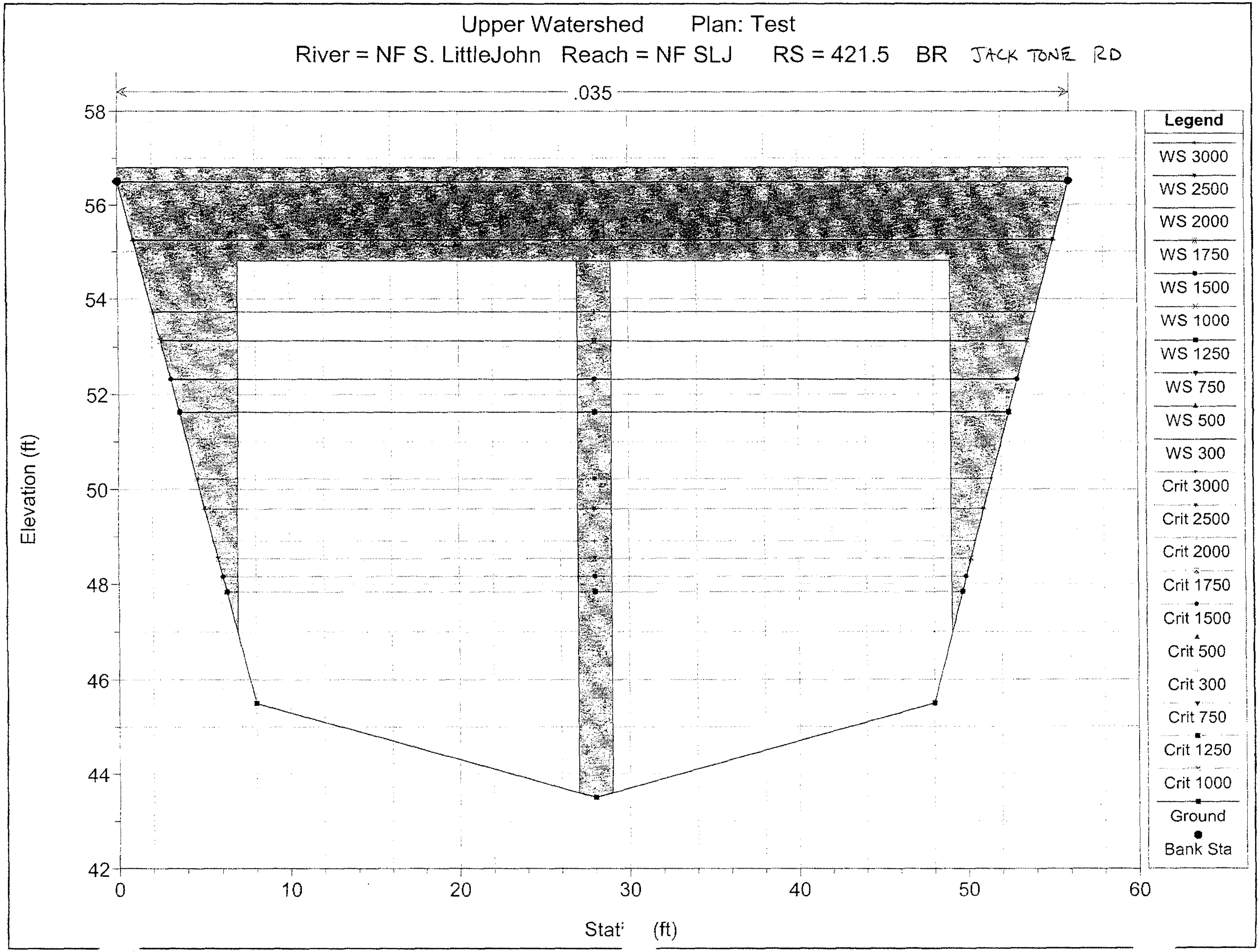
→ 99

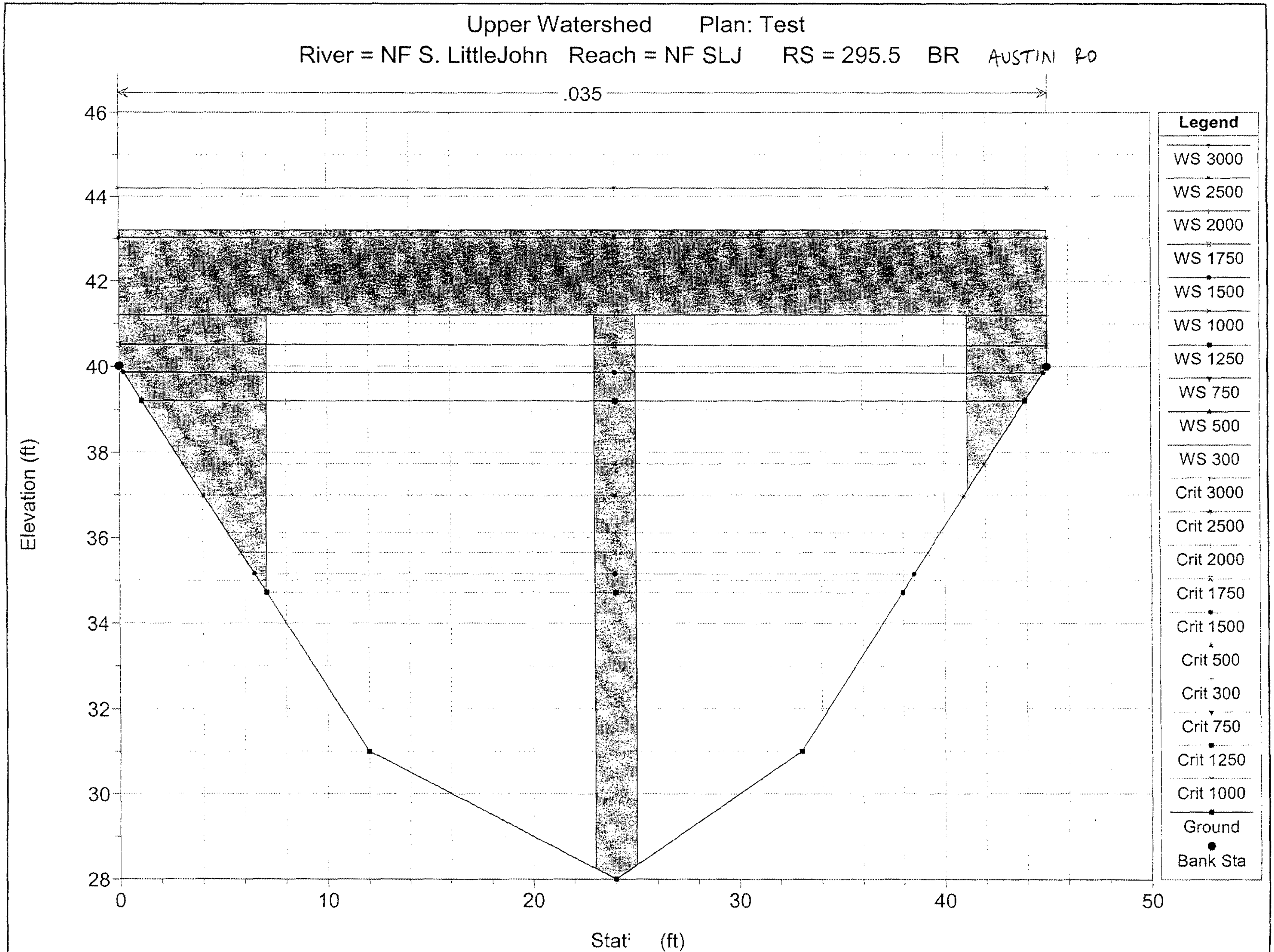
ELEV	AREA	AVG DEPTH	STORAGE	OUTFLOW
30	36	2	72	5
31	76	2.5	190	15
32	115	3	345	400 → OVER FRENCH CAMP 0.75' 1700' WEIR
33	135	3.5	475	2000

UNDER 99?

NORTH FORK SOUTH LITTLEJOHNS CREEK
@ JACK TONE ROAD

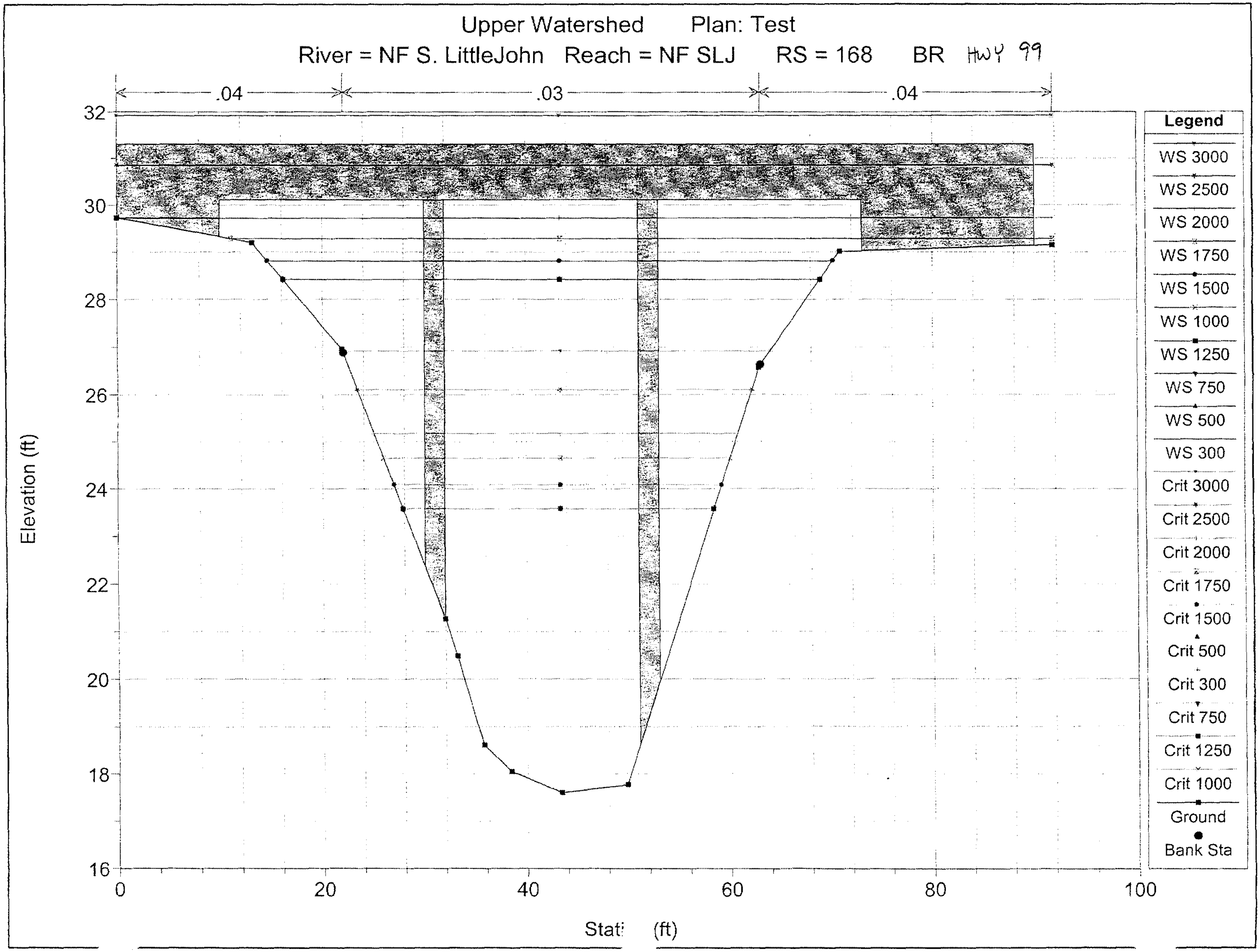



Upper Watershed Plan: Test
River = NF S. LittleJohn Reach = NF SLJ RS = 421.5 BR JACK TONE RD
.035
Elevation (ft)
Stat' (ft)
58
56
54
52
50
48
46
44
42
0
10
20
30
40
50
60
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1000
WS 1250
WS 750
WS 500
WS 300
Crit 3000
Crit 2500
Crit 2000
Crit 1750
Crit 1500
Crit 500
Crit 300
Crit 750
Crit 1250
Crit 1000
Ground
Bank Sta
214



Upper Watershed Plan: Test
River = NF S. LittleJohn Reach = NF SLJ RS = 295.5 BR AUSTIN RD
.035
Elevation (ft)
46
44
42
40
38
36
34
32
30
28
0
10
20
30
40
50
Stat' (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1000
WS 1250
WS 750
WS 500
WS 300
Crit 3000
Crit 2500
Crit 2000
Crit 1750
Crit 1500
Crit 500
Crit 300
Crit 750
Crit 1250
Crit 1000
Ground
Bank Sta

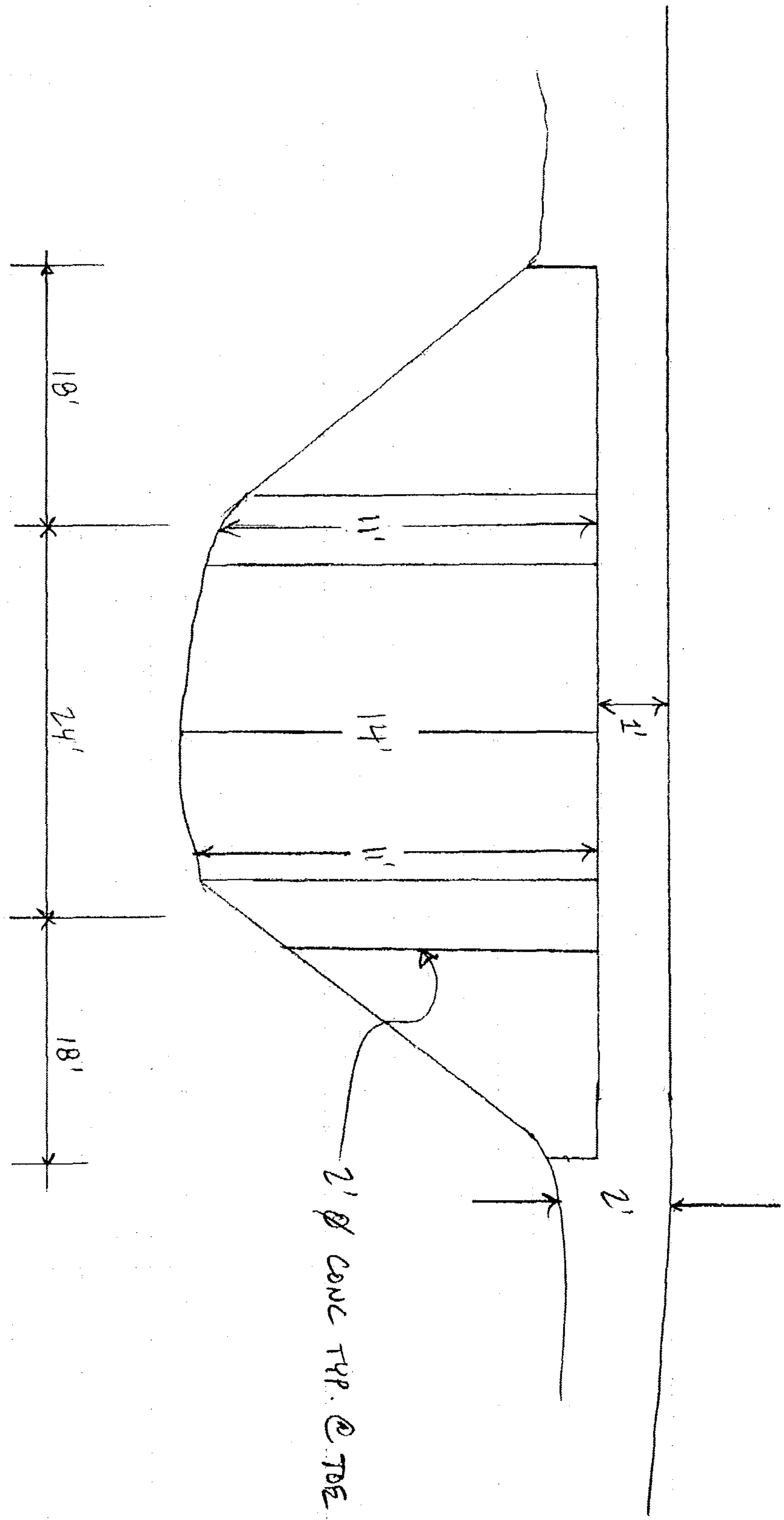
215

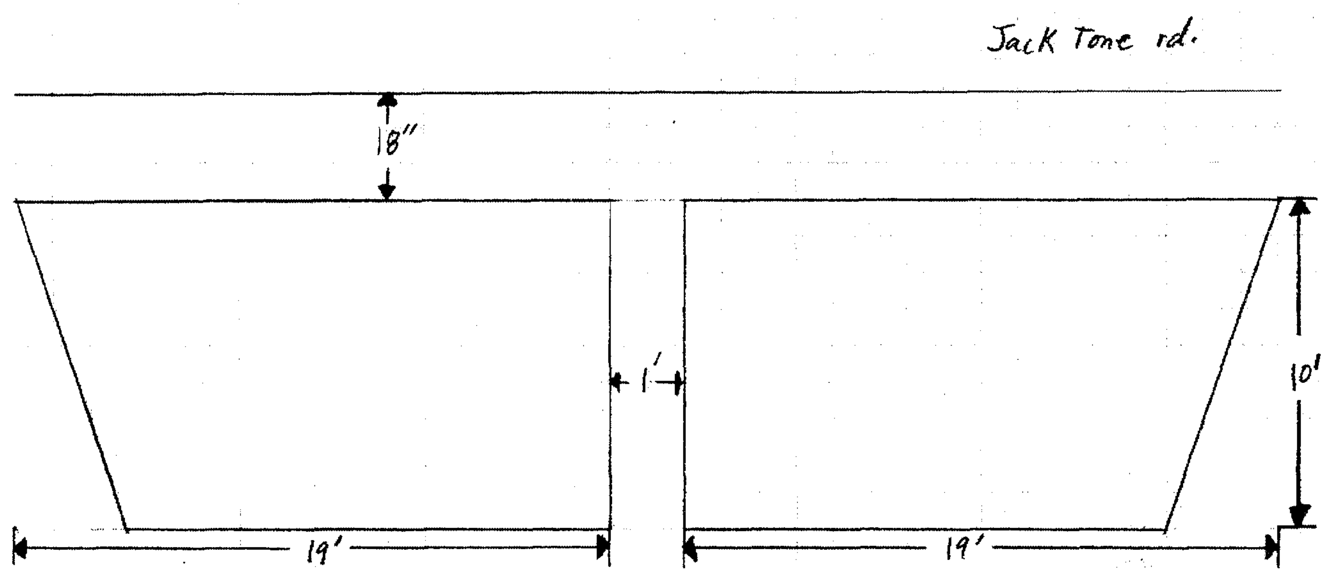


Upper Watershed Plan: Test
River = NF S. LittleJohn Reach = NF SLJ RS = 168 BR HWY 99
.04
.03
.04
Elevation (ft)
Stati (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1000
WS 1250
WS 750
WS 500
WS 300
Crit 3000
Crit 2500
Crit 2000
Crit 1750
Crit 1500
Crit 500
Crit 300
Crit 750
Crit 1250
Crit 1000
Ground
Bank Sta

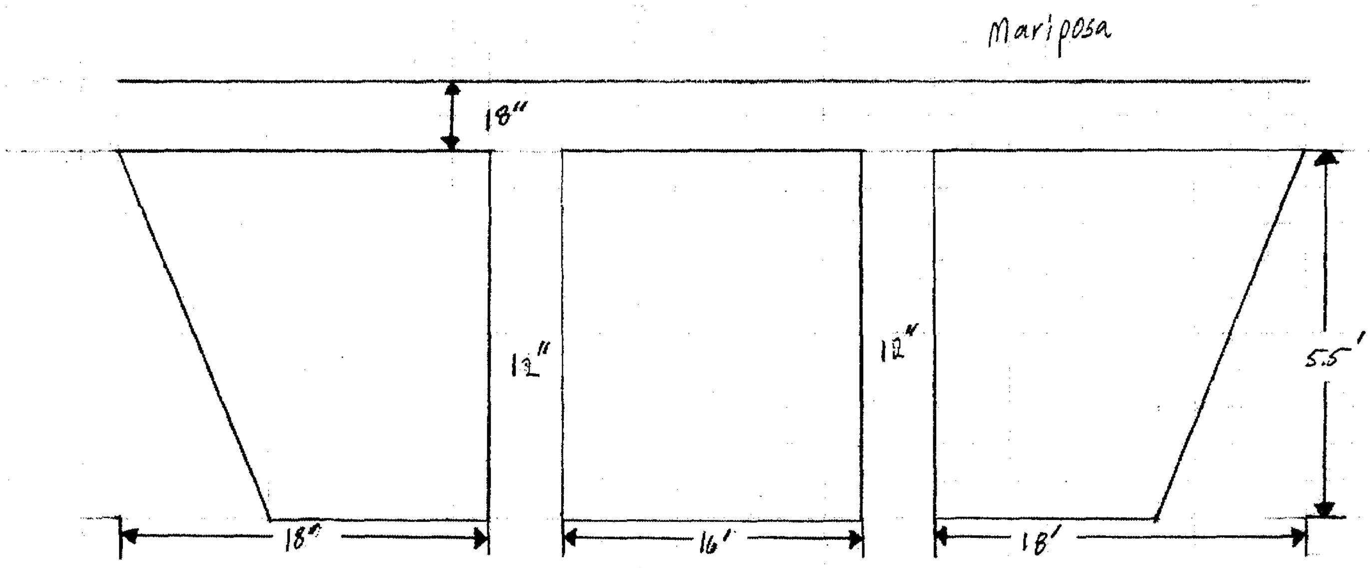
216

S. FORK S. LITTLEJOHNS
@ AUSTIN ROAD

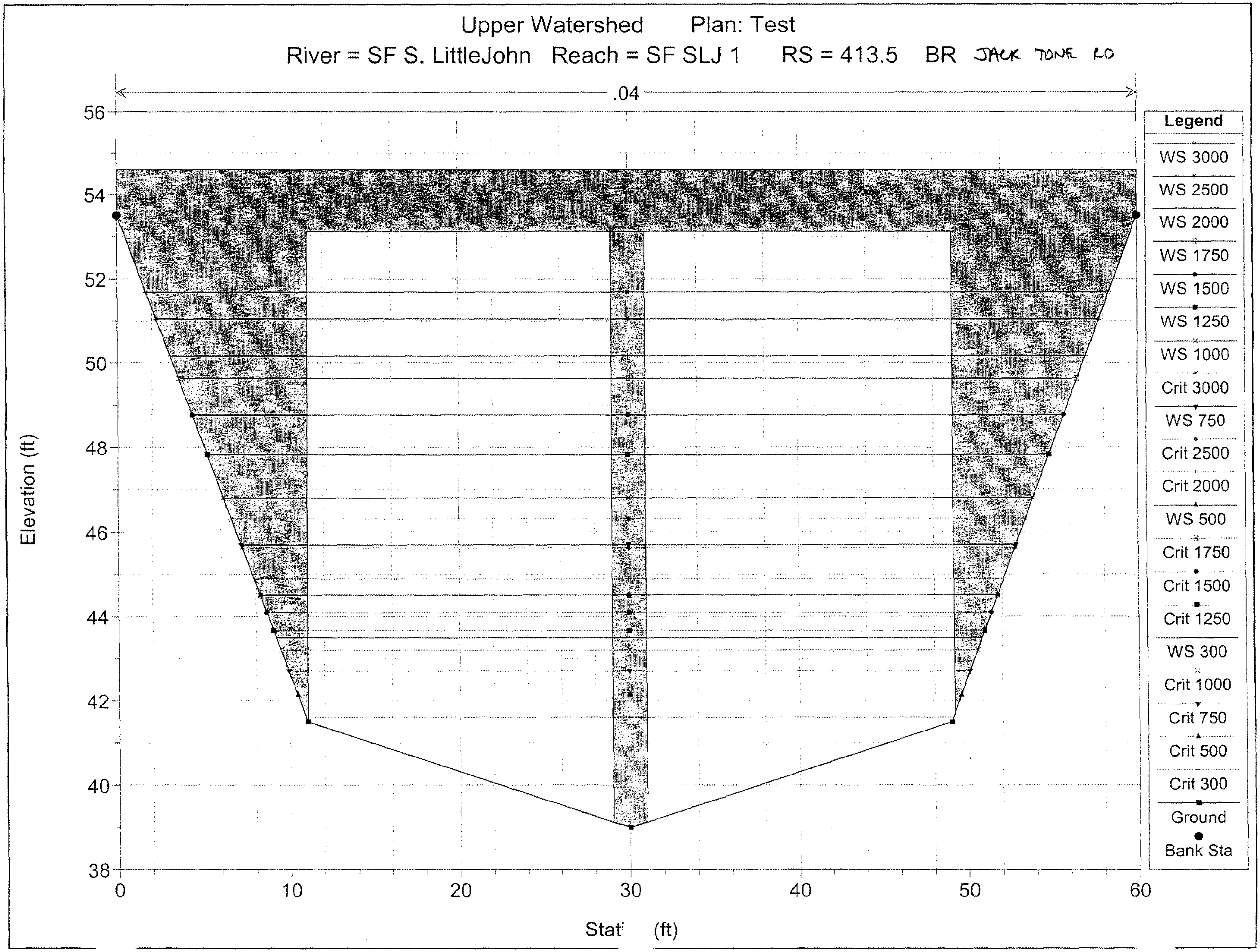


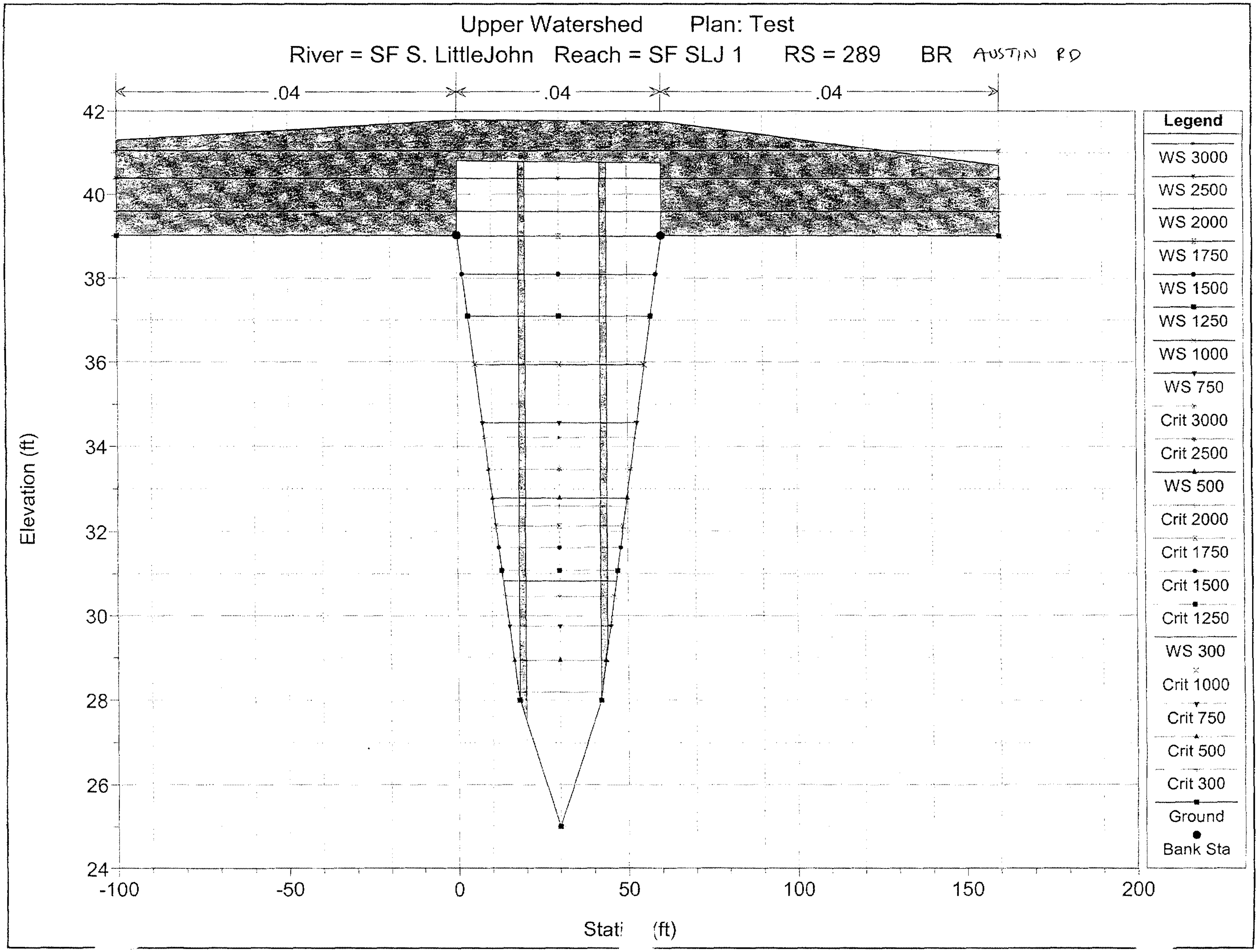


~~South Fork~~ South Littlejohns Creek and Jack Tone rd



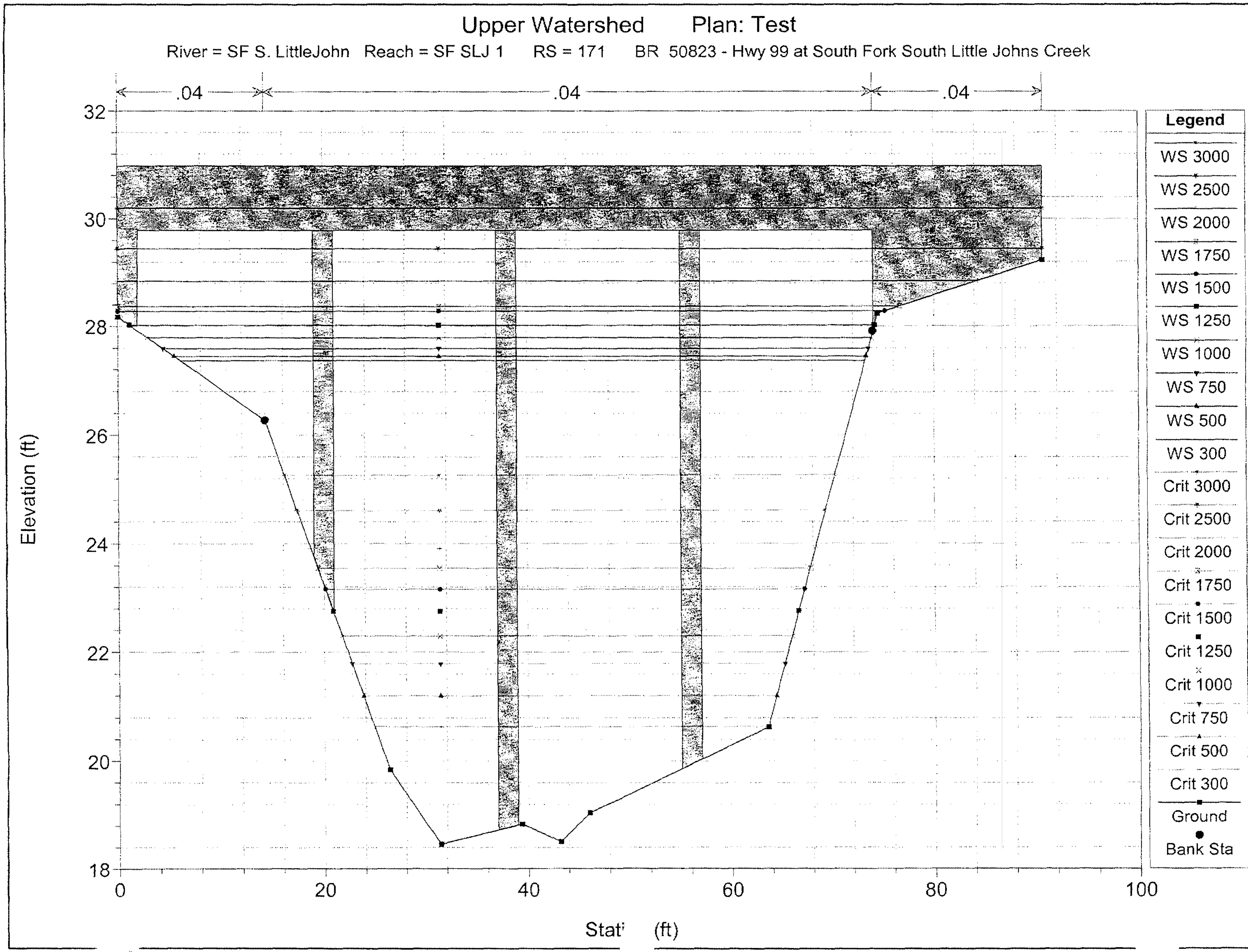
S. Littlejohn Creek and Mariposa


Upper Watershed Plan: Test
River = SF S. LittleJohn Reach = SF SLJ 1 RS = 413.5 BR JACK TONE RD
.04
Elevation (ft)
Station (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
Crit 3000
WS 750
Crit 2500
Crit 2000
WS 500
Crit 1750
Crit 1500
Crit 1250
WS 300
Crit 1000
Crit 750
Crit 500
Crit 300
Ground
Bank Sta
220



Upper Watershed Plan: Test
River = SF S. LittleJohn Reach = SF SLJ 1 RS = 289 BR AUSTIN RD
.04
.04
.04
Elevation (ft)
Stati (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
WS 750
Crit 3000
Crit 2500
WS 500
Crit 2000
Crit 1750
Crit 1500
Crit 1250
WS 300
Crit 1000
Crit 750
Crit 500
Crit 300
Ground
Bank Sta

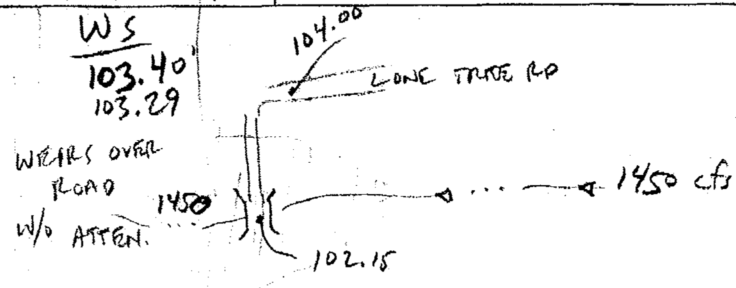
221



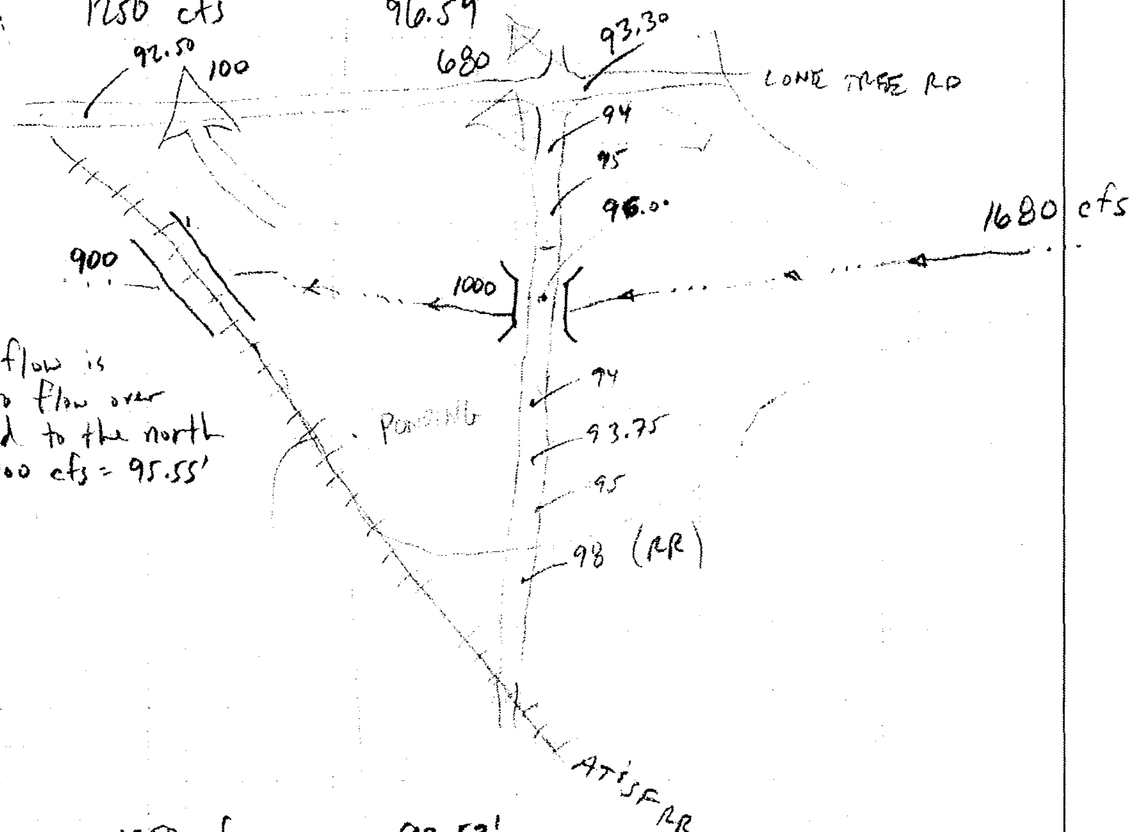
Upper Watershed Plan: Test
River = SF S. LittleJohn Reach = SF SLJ 1 RS = 171 BR 50823 - Hwy 99 at South Fork South Little Johns Creek
.04
.04
.04
Elevation (ft)
Stat (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
WS 750
WS 500
WS 300
Crit 3000
Crit 2500
Crit 2000
Crit 1750
Crit 1500
Crit 1250
Crit 1000
Crit 750
Crit 500
Crit 300
Ground
Bank Sta

LONE TREE CREEK | CROSSING RATING TABLE

			WS
RS 877	BRENNAN RD:	1750 cfs	103.40'
		1450 cfs	103.29



RS 812	SEXTON RD:	1250 cfs	96.59'



AT 1250 cfs, flow is deep enough to flow over Lone Tree road to the north 3' Deep. 1000 cfs = 95.55'

RS 808	AT&SF RR	1250 cfs	93.53'
		1000 cfs	93.27'

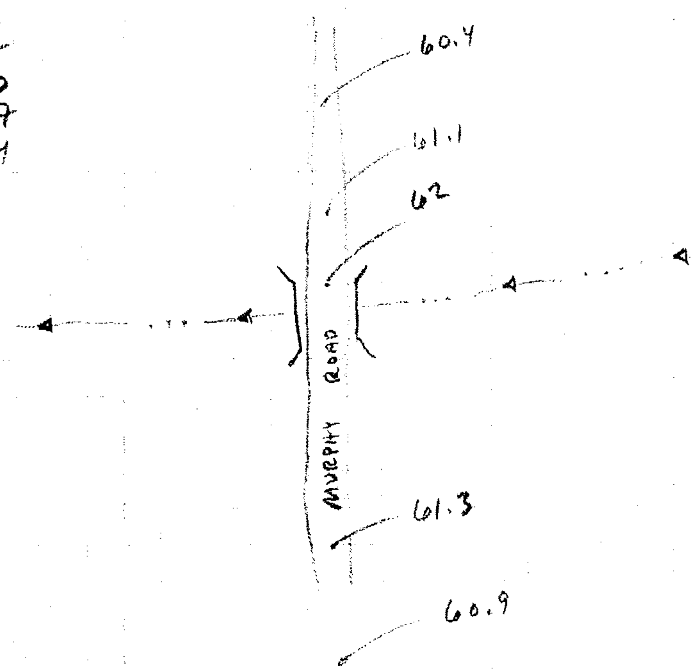
NEGLIGIBLE STORAGE; EXCESS FLOW IS DIVERTED TO NORTH BEHIND RR
SEXTON + AT&SF RR COMBINED INFLOW-DIV TABLE

IN	DIV
0	0
800	0
1000	150
1200	330
1500	600
2000	1050

LONE TREE CREEK | CROSSINGS RATING TABLES

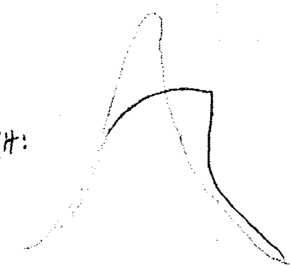
RS 560 MURPHY RD:

	WS
1500	62.82
1250	62.70
1000	62.47
750	61.94



ELEV	AREA	STORAGE	OUTFLOW
60.05	0.	0	500
61.94	580	500	750
62.47	800	650	1000
62.7	850	675	1250
62.8	900	700	1500

RESULTS IN HYDROGRAPH:



USE DIVERSION TO METERED RES TO SIMULATE OVERBANK STORAGE

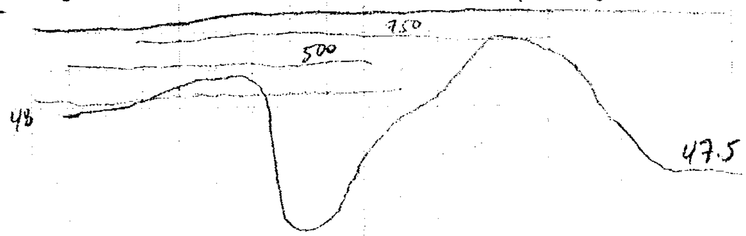
INFLOW	DIV
0	0
500	0
750	150
1000	350
1250	550
1500	700

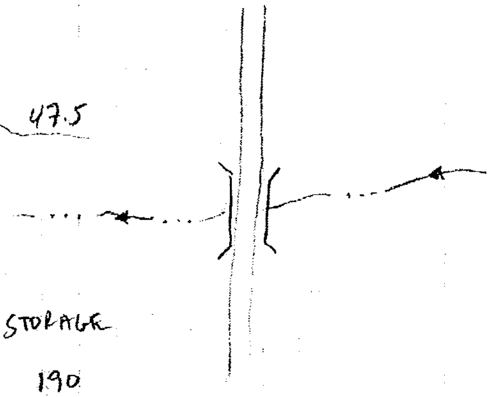
→

STORAGE	DISCHARGE
0	0
100	10
300	50
600	100
700	120

STAEDTLER® No. 937 811E Engineer's Computation Pad

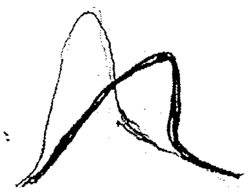
RS 450 JACK TONE RD. RD = 50.95





Q	ELEV	AREA	AVG. DEPTH	STORAGE
500	49.2	190	1	190
750	49.8	280	1.5	420
1000	50.9	645	2	1300
1250	51.3	790	2.2	1740
1500	51.6	940	2.4	2300

AGAIN, USE DIV TO RES TO AVOID THIS HYDROGRAPH:

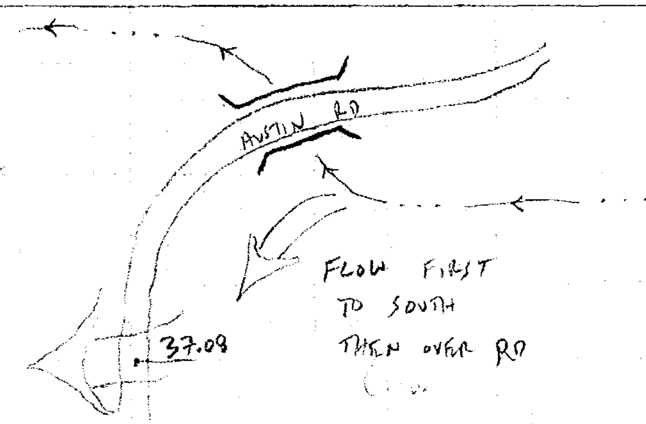


IN	DIV	STOR.	OUTFLOW
0	0	0	0
500	50	190	10
750	200	420	20
1000	350	1300	40
1250	475	1740	60
1500	600	2300	70

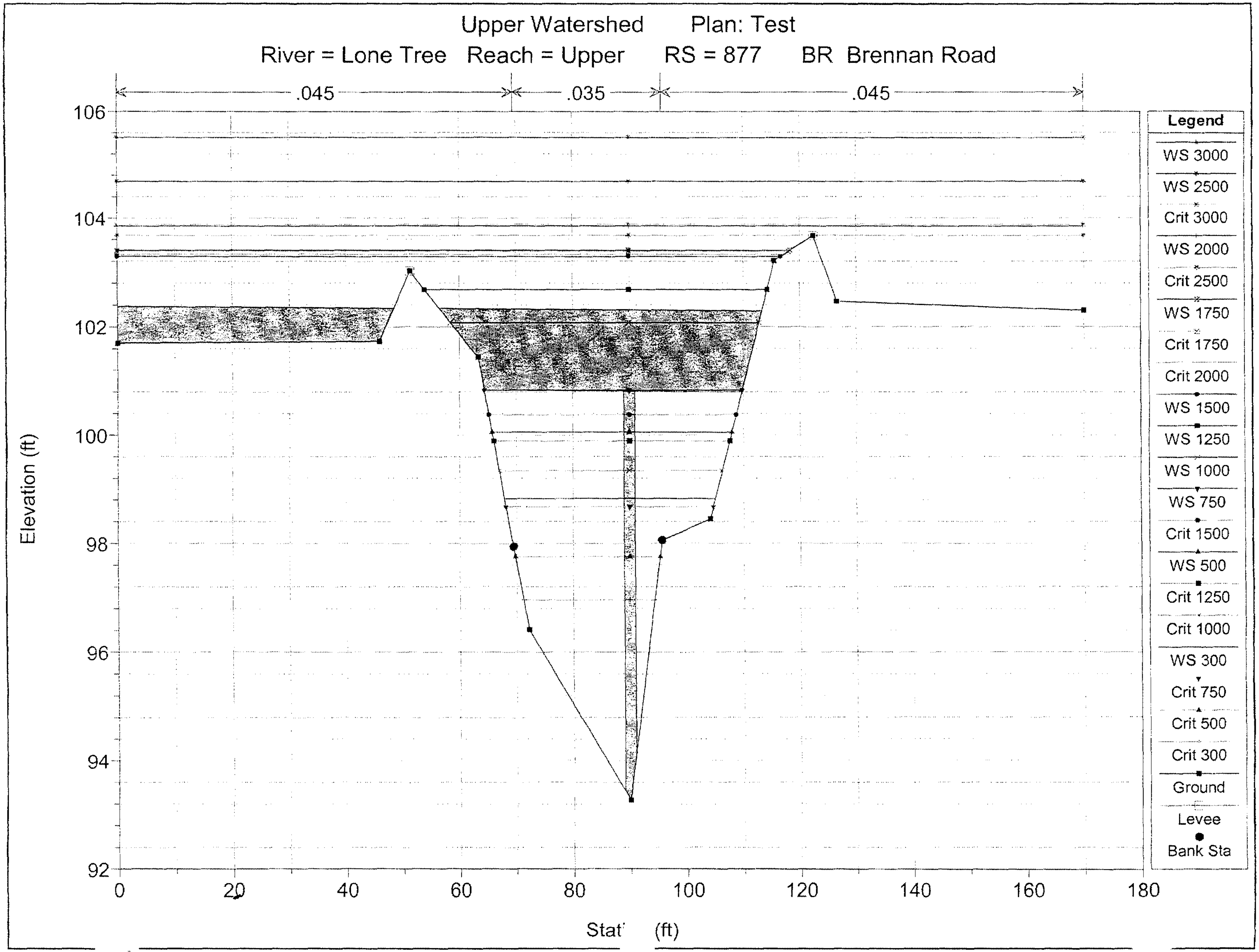
RS 302.5 AUSTIN RD

DECK ELEV = 39.4

Q	ELEV	AREA	DEPTH	STORAGE
1250	37	12.5	0.1	1.3
1500	38.6	32	1	32
1750	38.9	49	1.1	55
2000	39.1	65	1.3	85
2500	39.5	83	1.5	125
3000	40.0	145	1.7	250

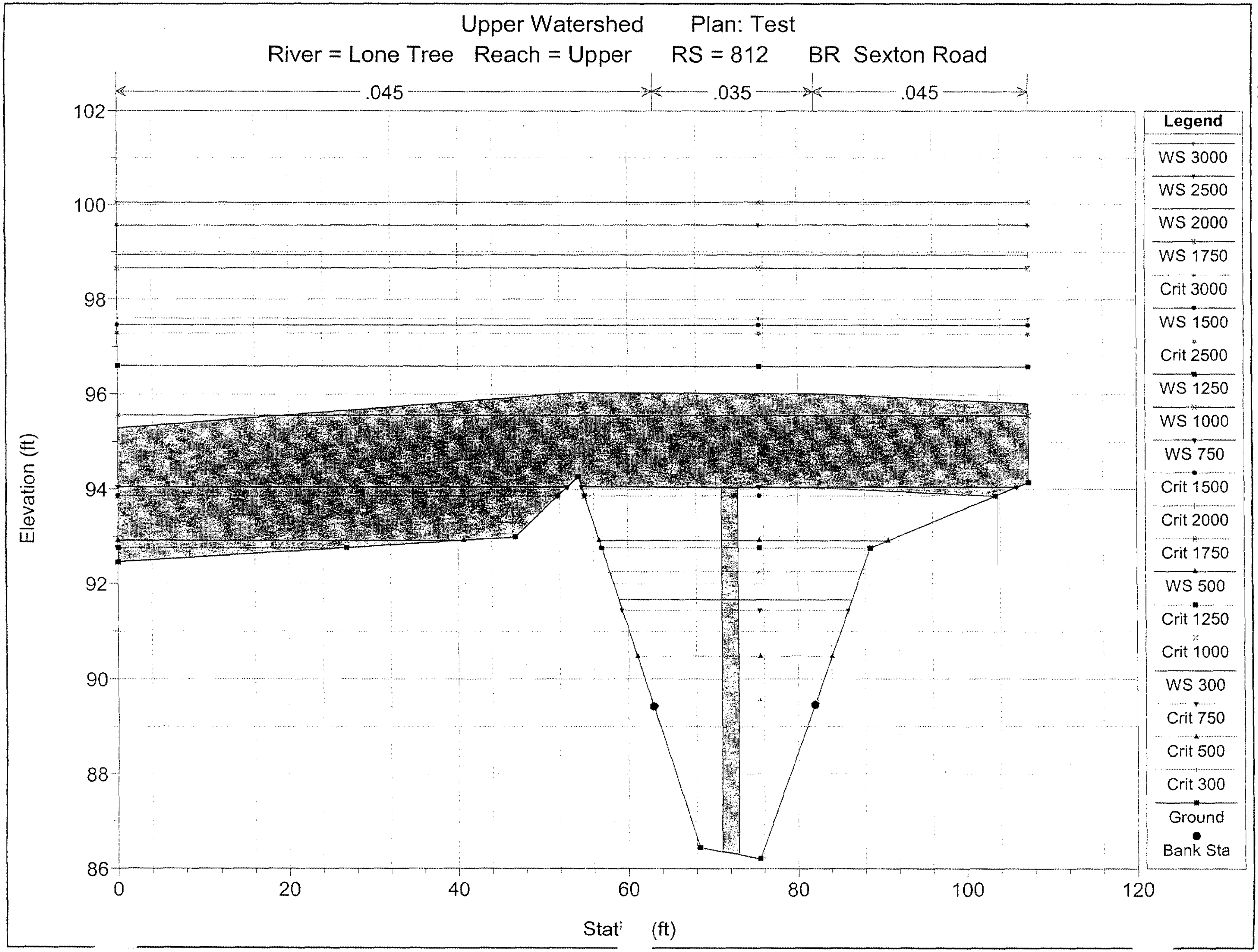


FLOW FIRST TO SOUTH THEN OVER RD

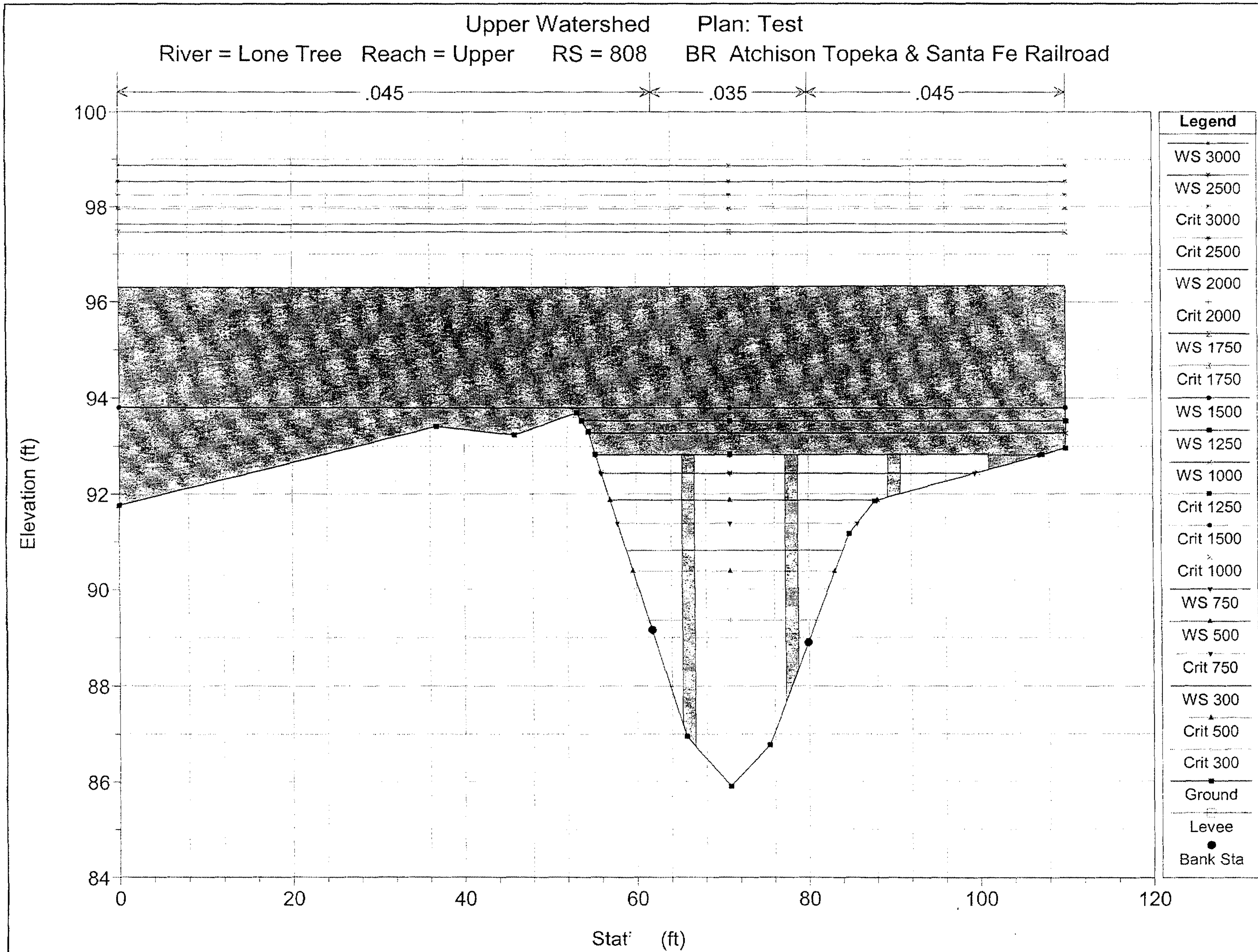


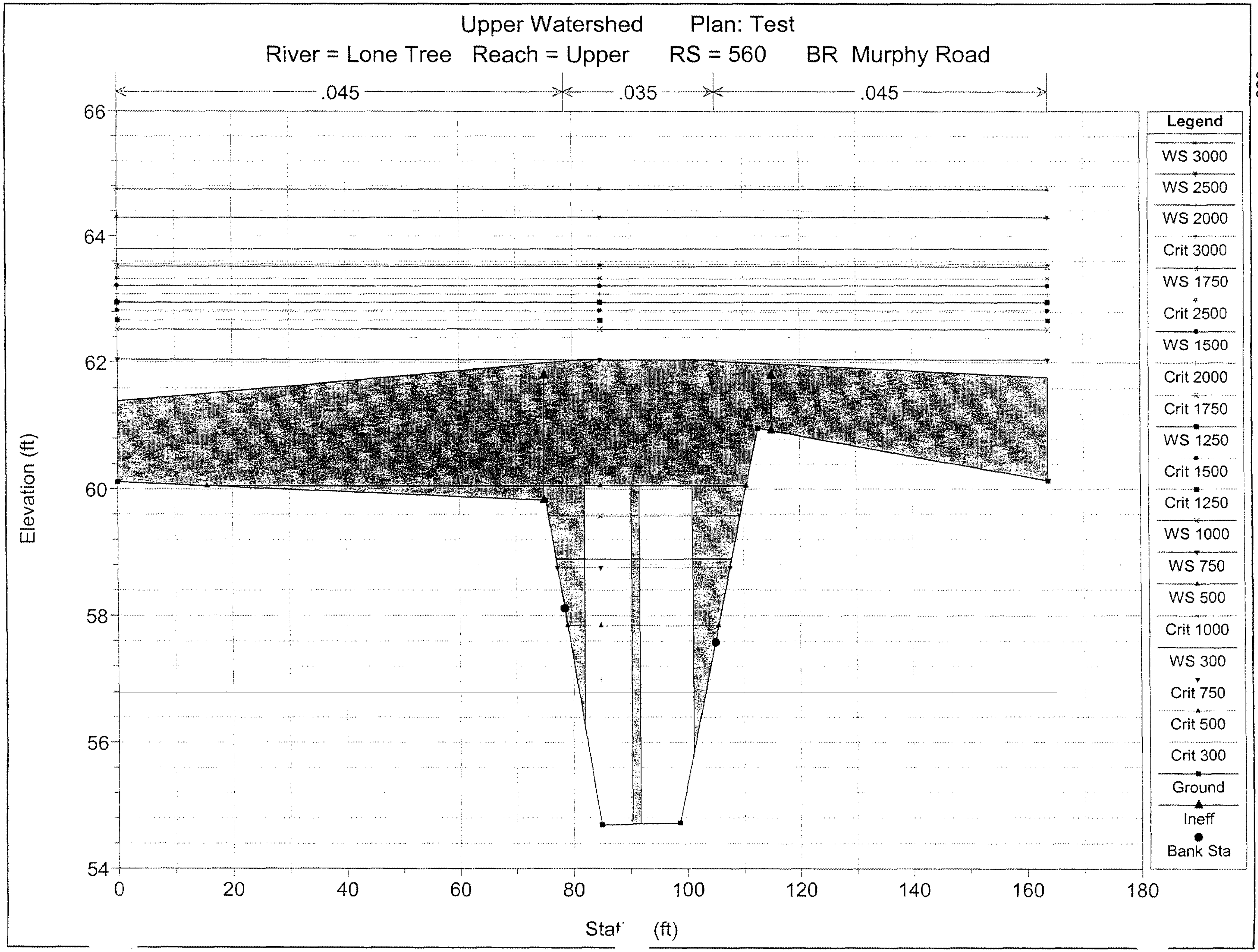
Upper Watershed Plan: Test
River = Lone Tree Reach = Upper RS = 877 BR Brennan Road
.045
.035
.045
Elevation (ft)
Stat (ft)
Legend
WS 3000
WS 2500
Crit 3000
WS 2000
Crit 2500
WS 1750
Crit 1750
Crit 2000
WS 1500
WS 1250
WS 1000
WS 750
Crit 1500
WS 500
Crit 1250
Crit 1000
WS 300
Crit 750
Crit 500
Crit 300
Ground
Levee
Bank Sta

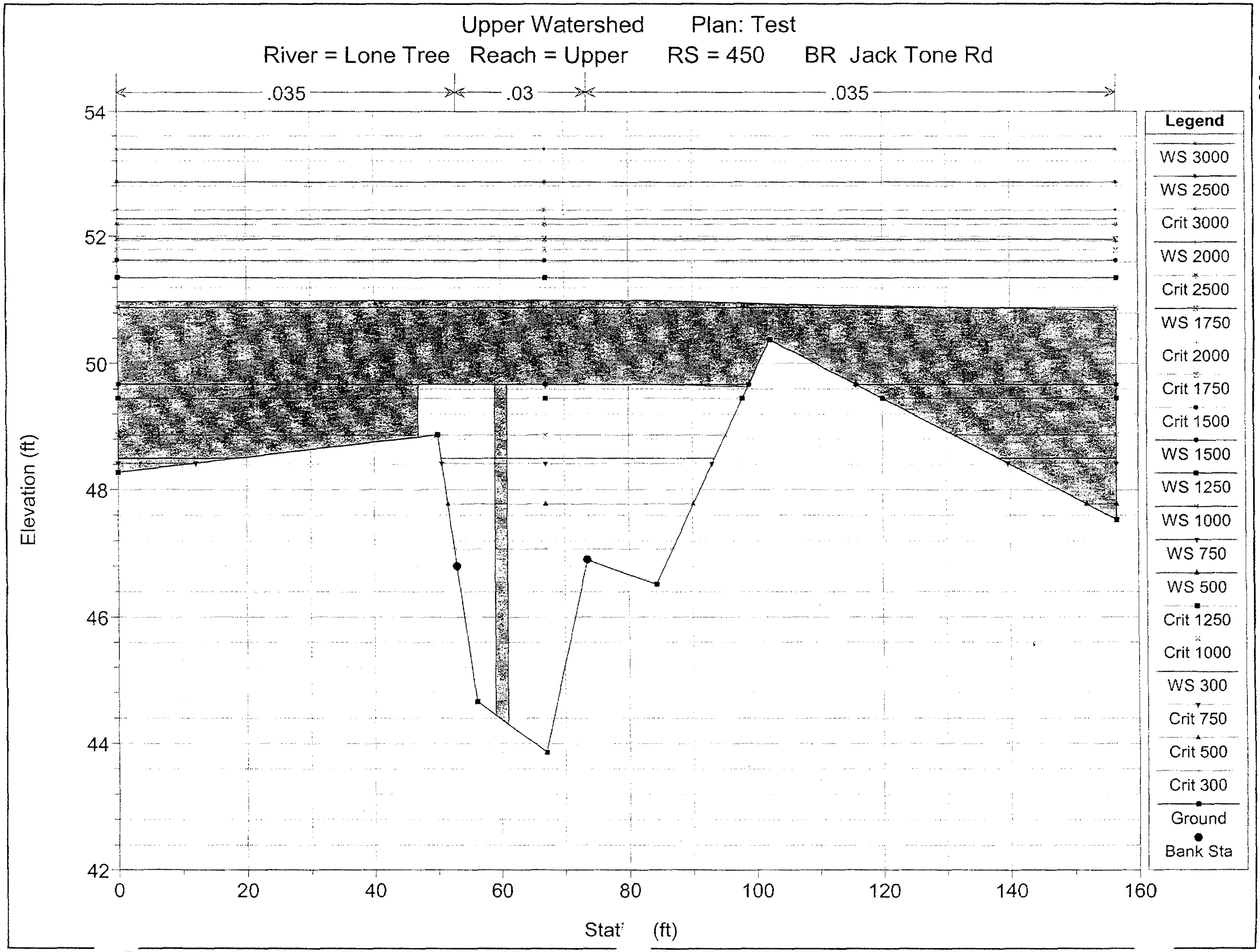
226



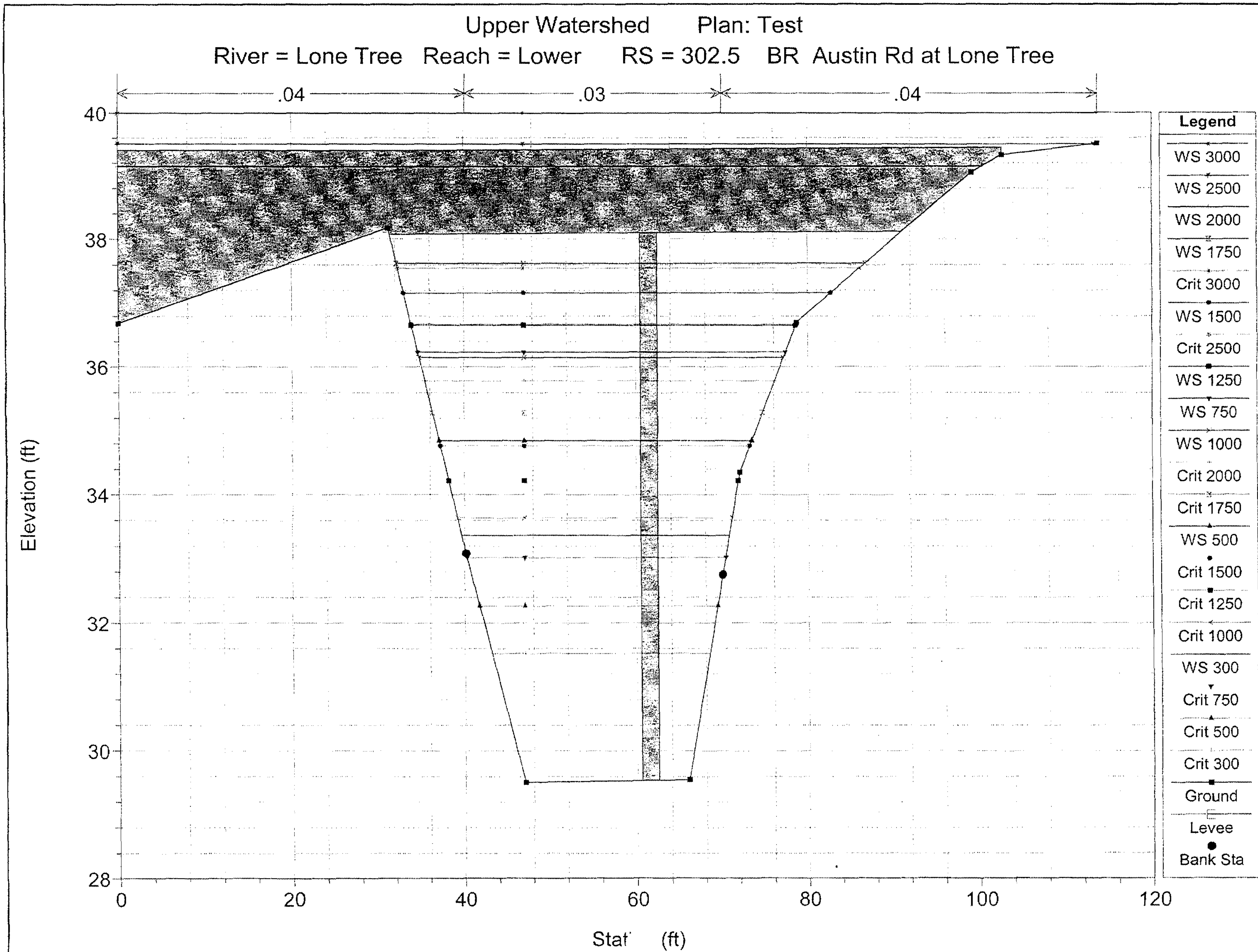
Upper Watershed Plan: Test
River = Lone Tree Reach = Upper RS = 812 BR Sexton Road
.045
.035
.045
Elevation (ft)
Stati (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
Crit 3000
WS 1500
Crit 2500
WS 1250
WS 1000
WS 750
Crit 1500
Crit 2000
Crit 1750
WS 500
Crit 1250
Crit 1000
WS 300
Crit 750
Crit 500
Crit 300
Ground
Bank Sta


Upper Watershed Plan: Test
River = Lone Tree Reach = Upper RS = 808 BR Atchison Topeka & Santa Fe Railroad
.045
.035
.045
Elevation (ft)
Stat' (ft)
Legend
WS 3000
WS 2500
Crit 3000
Crit 2500
WS 2000
Crit 2000
WS 1750
Crit 1750
WS 1500
WS 1250
WS 1000
Crit 1250
Crit 1500
Crit 1000
WS 750
WS 500
Crit 750
WS 300
Crit 500
Crit 300
Ground
Levee
Bank Sta


Upper Watershed Plan: Test
River = Lone Tree Reach = Upper RS = 560 BR Murphy Road
.045
.035
.045
Legend
WS 3000
WS 2500
WS 2000
Crit 3000
WS 1750
Crit 2500
WS 1500
Crit 2000
Crit 1750
WS 1250
Crit 1500
Crit 1250
WS 1000
WS 750
WS 500
Crit 1000
WS 300
Crit 750
Crit 500
Crit 300
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
66
64
62
60
58
56
54
0
20
40
60
80
100
120
140
160
180

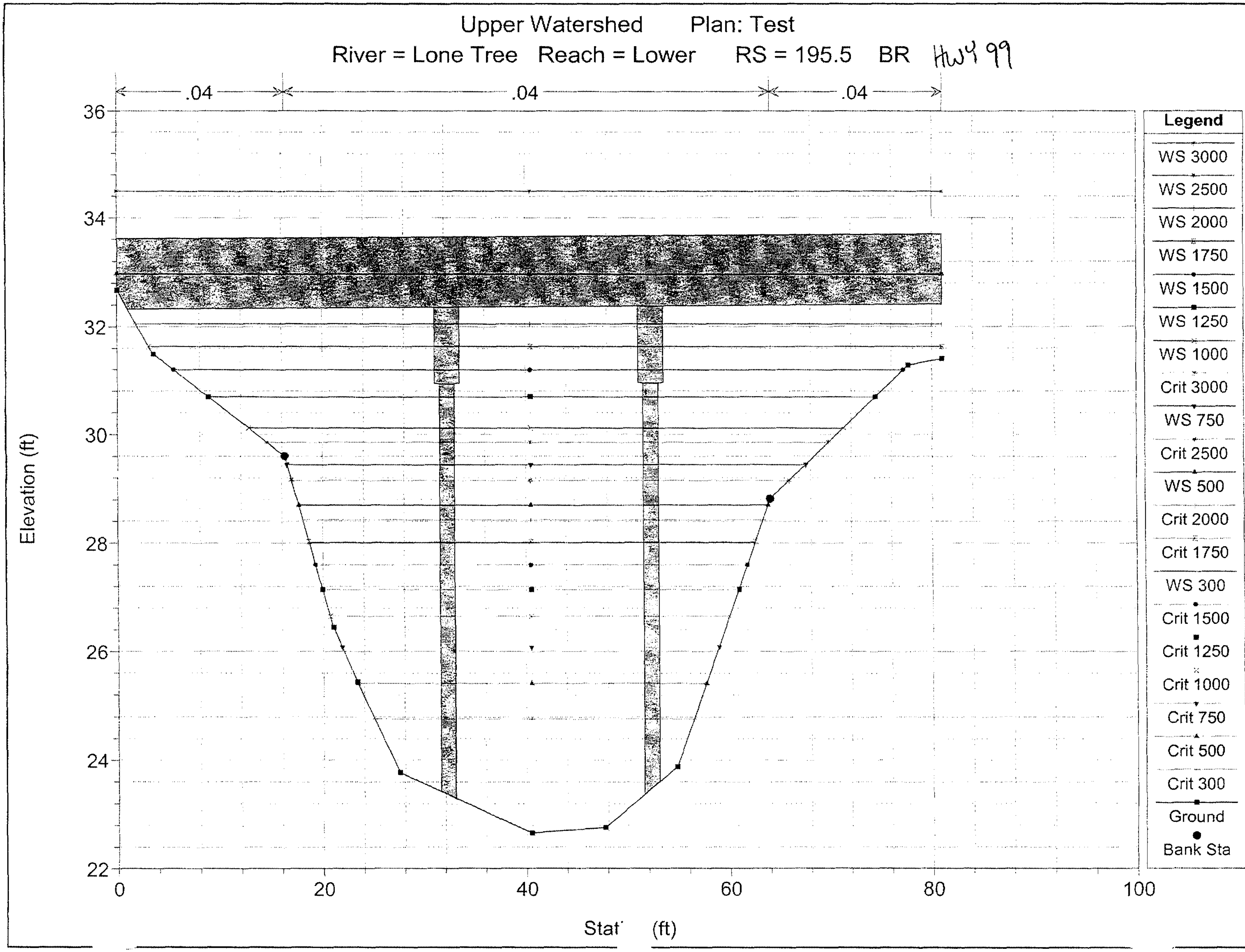


Upper Watershed Plan: Test
River = Lone Tree Reach = Upper RS = 450 BR Jack Tone Rd
.035
.03
.035
Legend
WS 3000
WS 2500
Crit 3000
WS 2000
Crit 2500
WS 1750
Crit 2000
Crit 1750
Crit 1500
WS 1500
WS 1250
WS 1000
WS 750
WS 500
Crit 1250
Crit 1000
WS 300
Crit 750
Crit 500
Crit 300
Ground
Bank Sta
Elevation (ft)
Stat (ft)



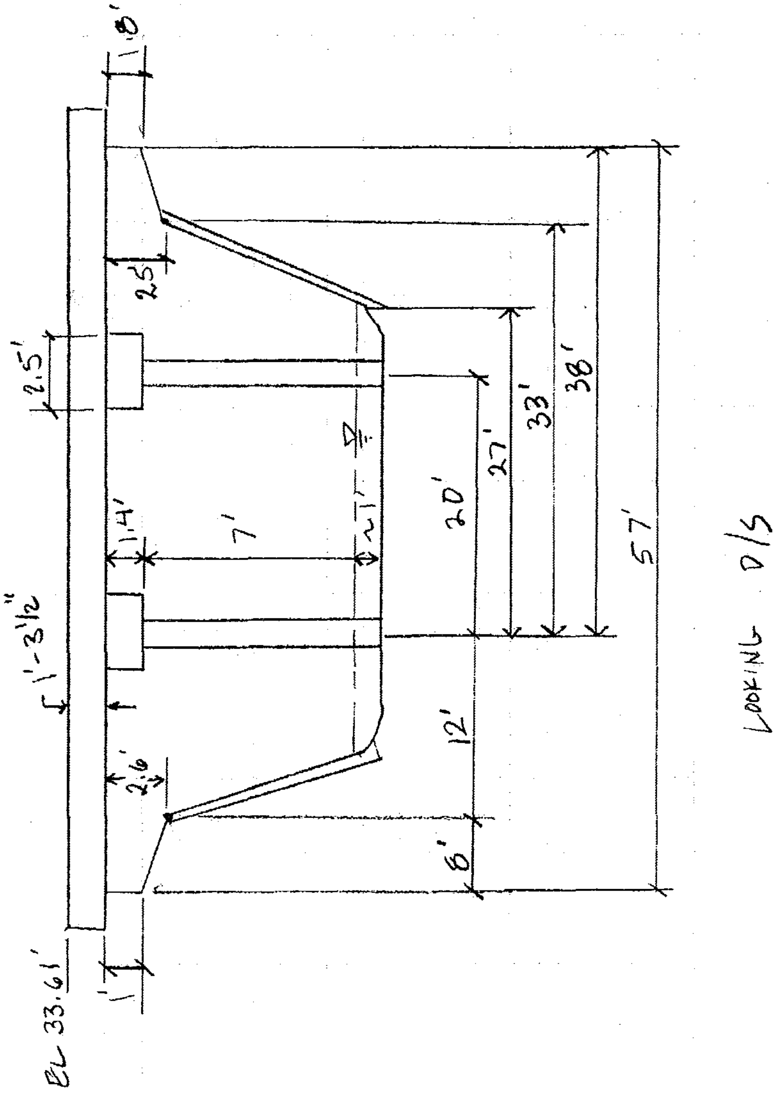
Upper Watershed Plan: Test
River = Lone Tree Reach = Lower RS = 302.5 BR Austin Rd at Lone Tree
.04
.03
.04
Elevation (ft)
Stat (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
Crit 3000
WS 1500
Crit 2500
WS 1250
WS 750
WS 1000
Crit 2000
Crit 1750
WS 500
Crit 1500
Crit 1250
Crit 1000
WS 300
Crit 750
Crit 500
Crit 300
Ground
Levee
Bank Sta

231



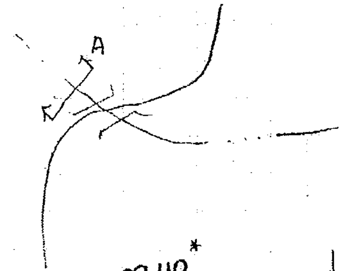
Upper Watershed Plan: Test
River = Lone Tree Reach = Lower RS = 195.5 BR
HWY 99
.04
.04
.04
Elevation (ft)
Stat' (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
Crit 3000
WS 750
Crit 2500
WS 500
Crit 2000
Crit 1750
WS 300
Crit 1500
Crit 1250
Crit 1000
Crit 750
Crit 500
Crit 300
Ground
Bank Sta

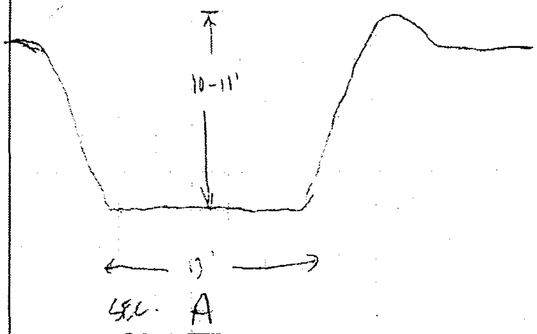
LONE TREE CK @ HIGHWAY 99 | 2/17/06

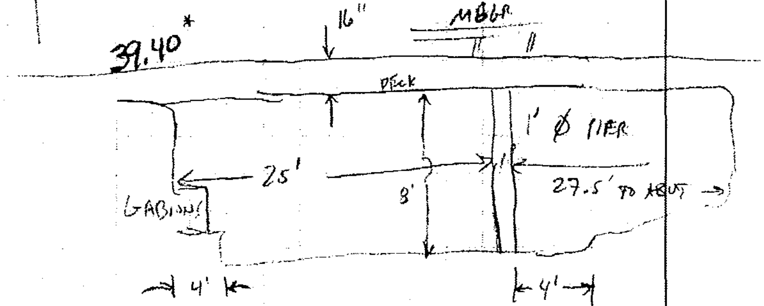


TELL KSN WHEN SHOOTING X-SEC, CONTINUE INTO FIELDS (OVER LEVEES, BERMS) 1/10/06

LONE TREE @ AUSTIN RD
PICS 1070 - 1075





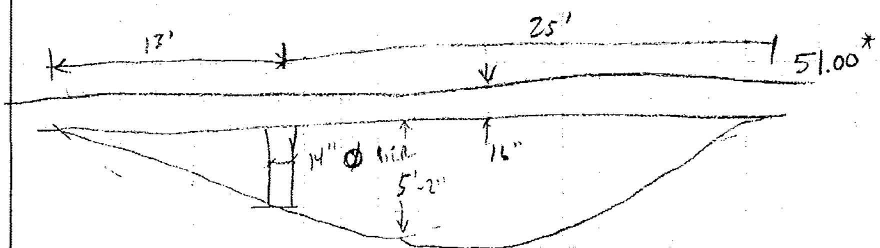


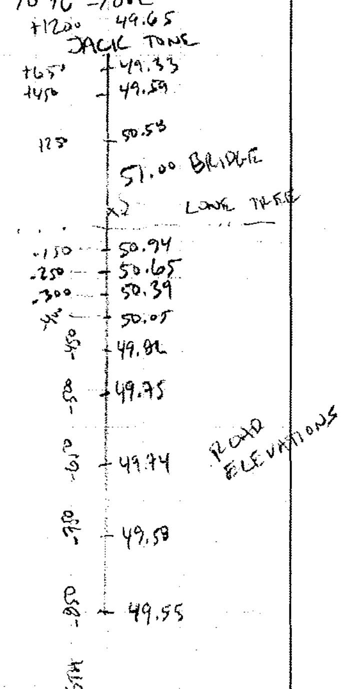
• VISIBLE HIGH WATER 24" BELOW BOTTOM OF DECK (JAN 1, 2006 STORM)

• SURVEY NEEDS: BRIDGE, 1 SEC 100' UPSTREAM
2 SEC 100', 200' DOWNSTREAM

LONE TREE @ JACK TONE RD — HIGH WATER 10" BELOW BOTTOM OF DECK

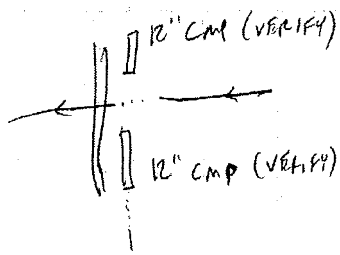
PICS 1076 - 1082





SURVEY: 3 SECTIONS: BRIDGE, 100' UP & D/S.

TEMPLE @ JACK TONE RD.
NO PICS



SURVEY
BRIDGE GEOM.
U/S, D/S 100'
ROAD
SIDE CULVERTS

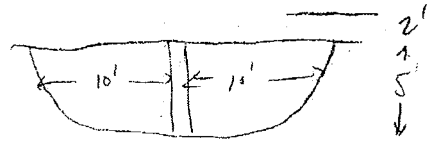
LONE TREE @ MURPHY RD DOUBLE BOX
18' x 6' 30° SKEW

SURVEY
BRIDGE GEOM.
U/S, D/S 100'
ROAD
SIDE CULVERT

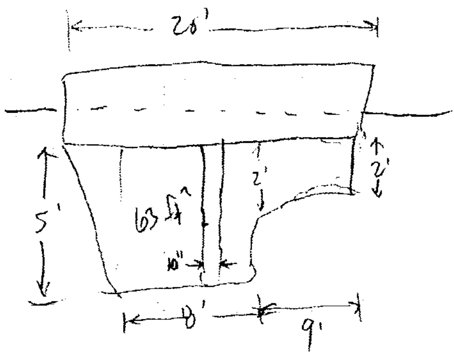
* FROM SIEGFRIED ROAD SURVEY

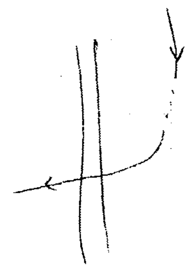
LONE TREE @ LONE TREE RD - HIGH WATER 10" BELOW BOTTOM OF DECK
PICS 1083-1085

LONE TREE @ CARROLTON
PICS 1086-88



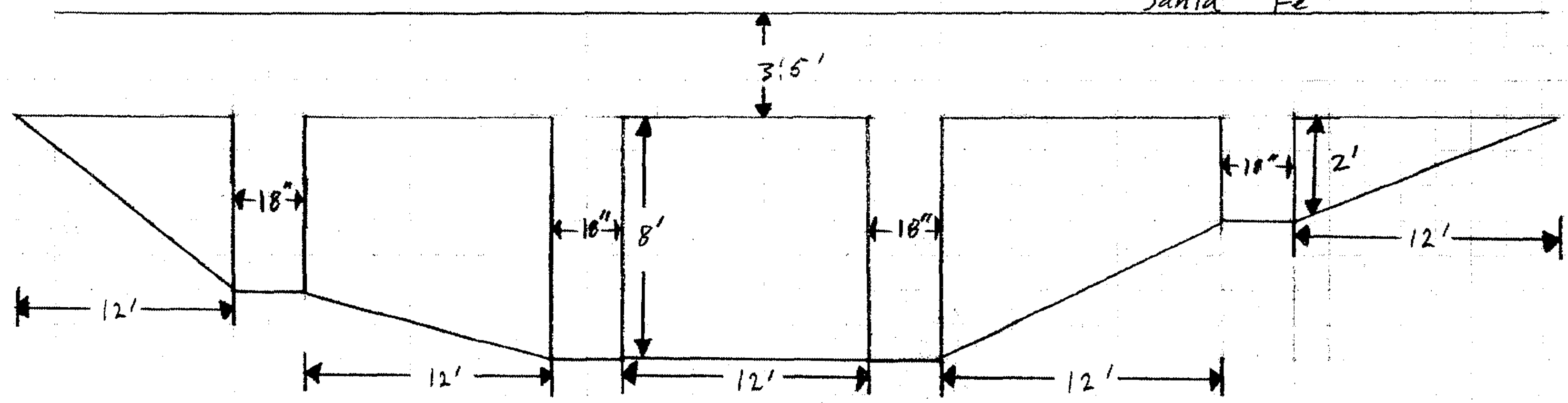
LONE TREE @ VAN ALLEN   63 sq. ft @ 6 ft/s = 380 cfs
PICS 89-95





A

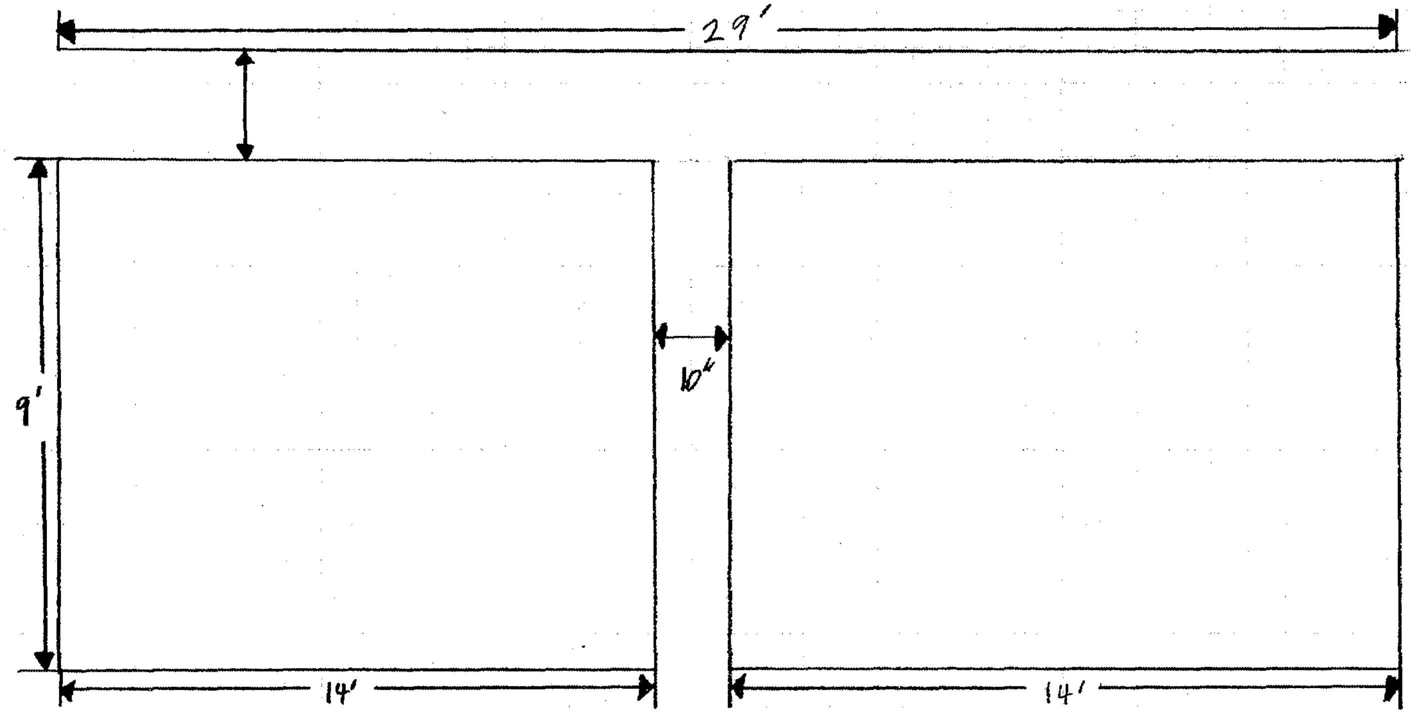
train Atchison Topeka and Santa Fe



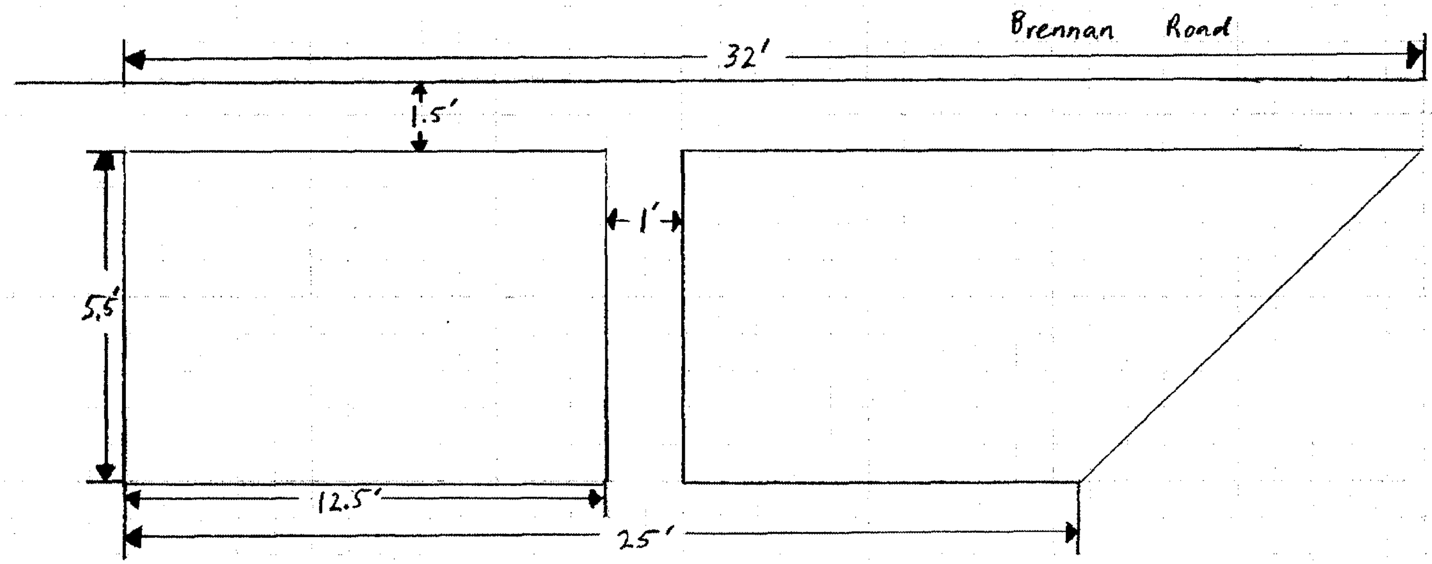
note 2 ft/sec
Adv. 1.25'

Lone Tree Creek and Atchison Topeka / Santa Fe

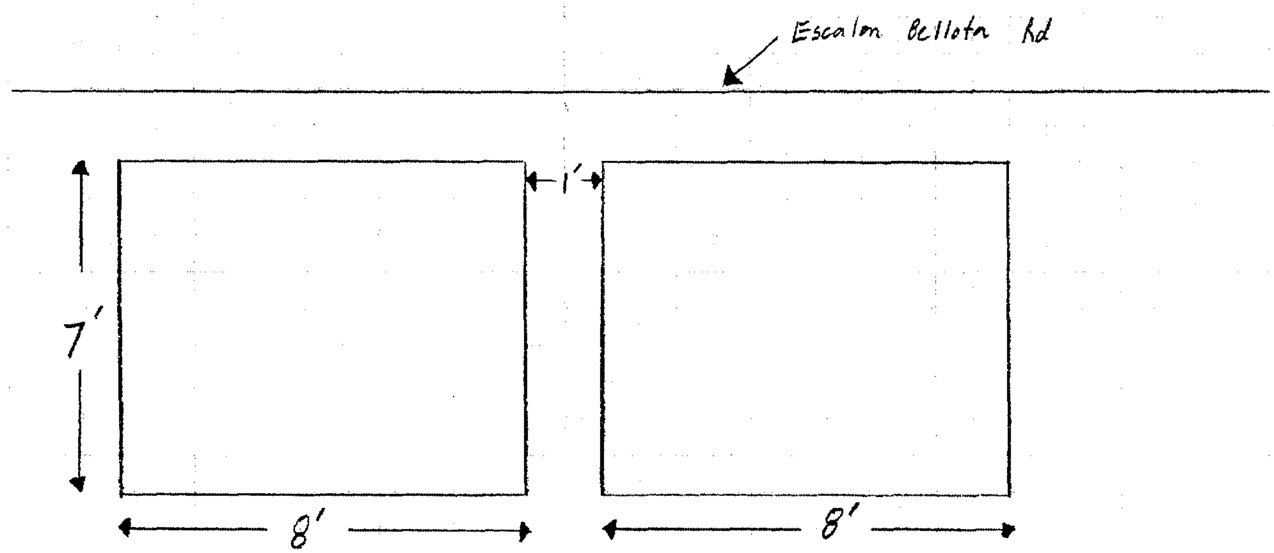
B



Lone Tree Creek and Sexton Rd.

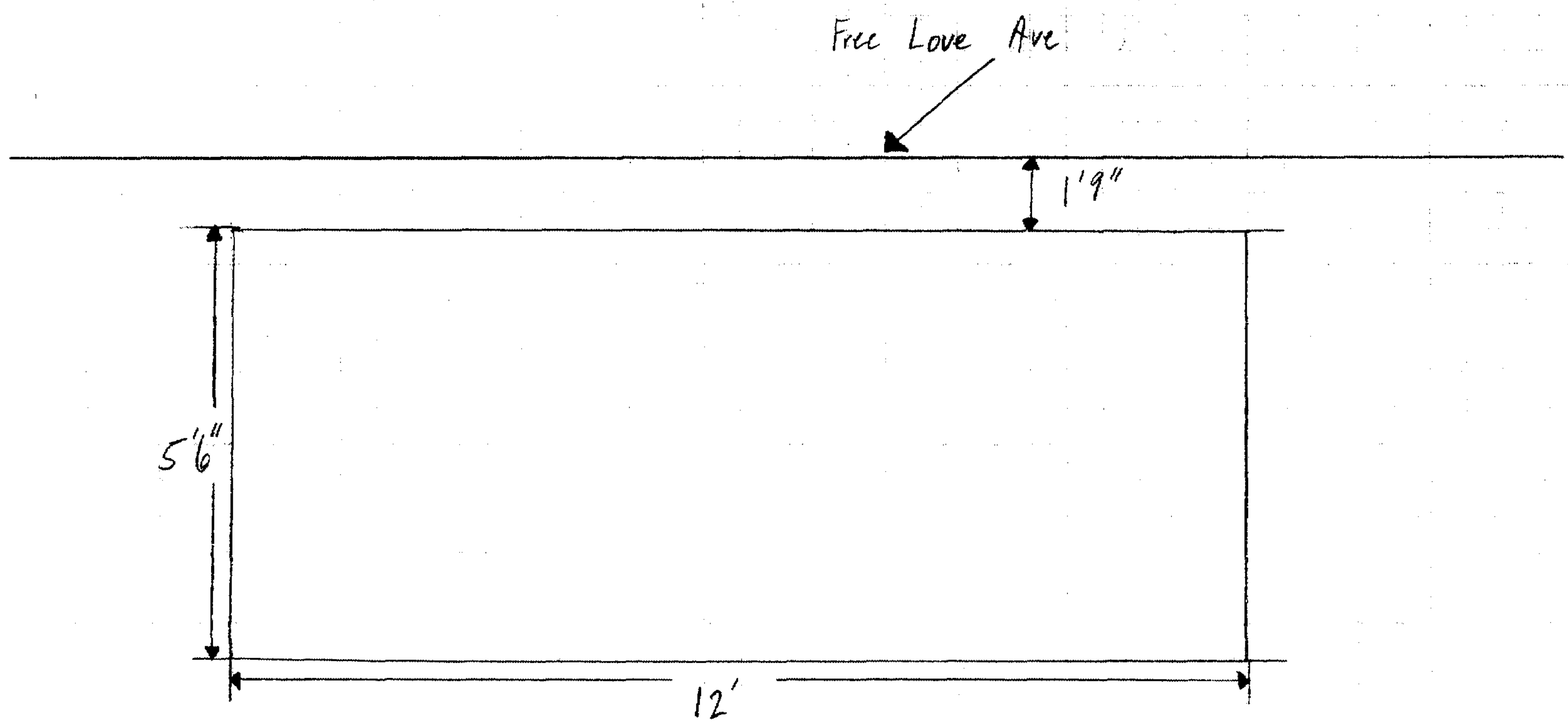


Lone Creek and Brennan Rd.



Lone Creek and Escalon Bellota rd.

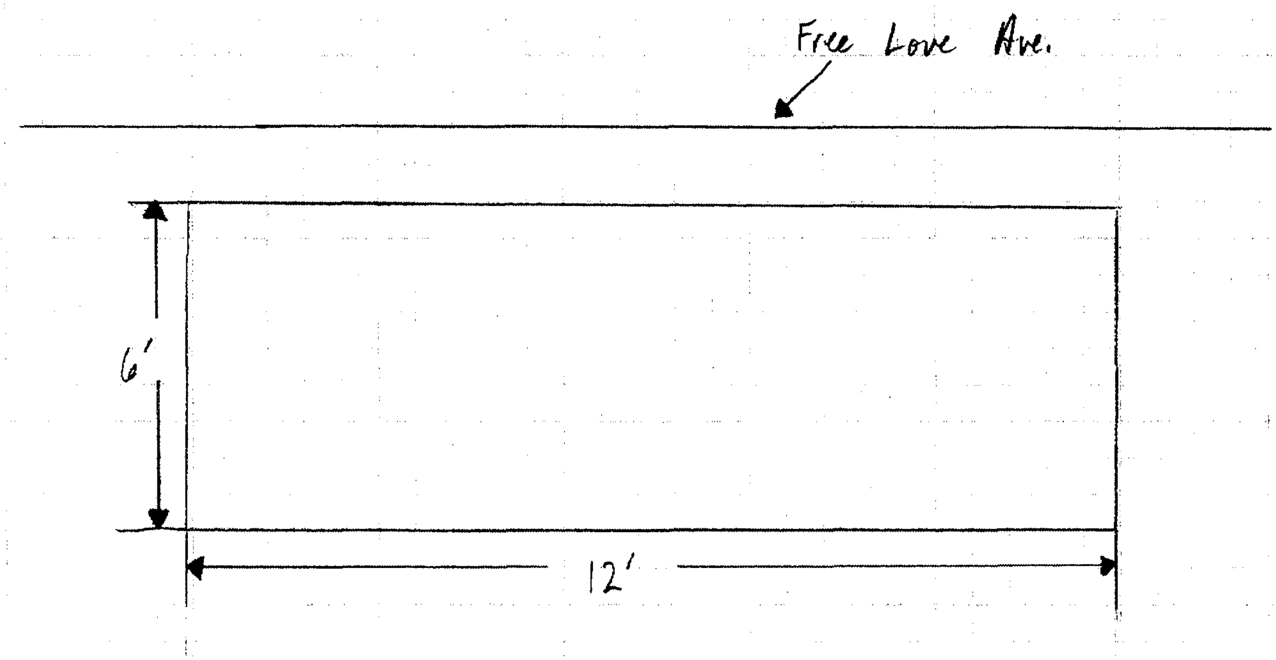
8



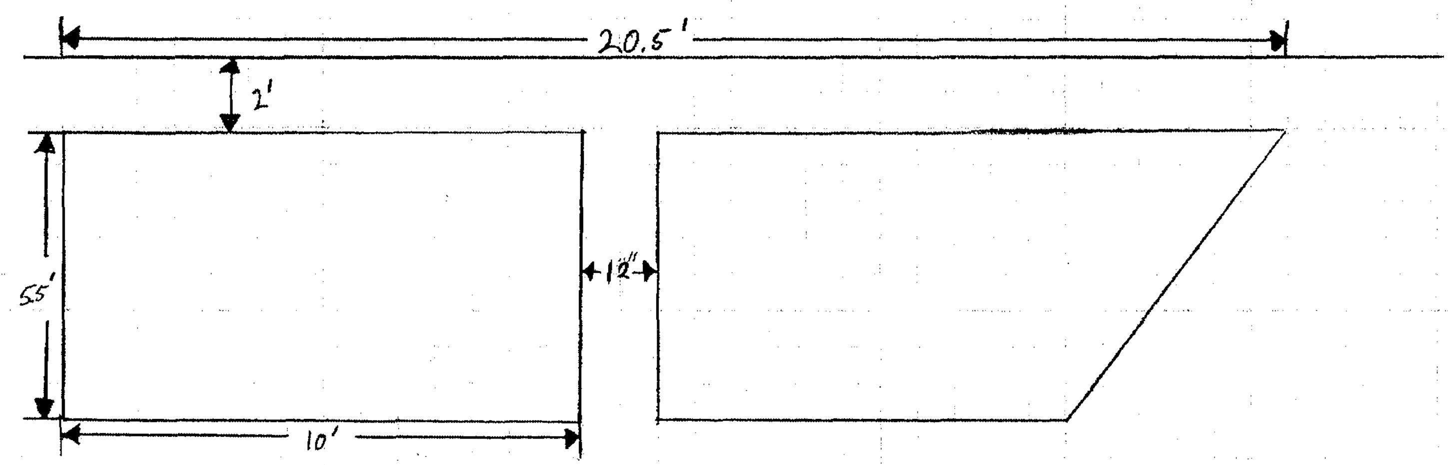
note: Sandy soil

Lone Creek and Free Love Ave

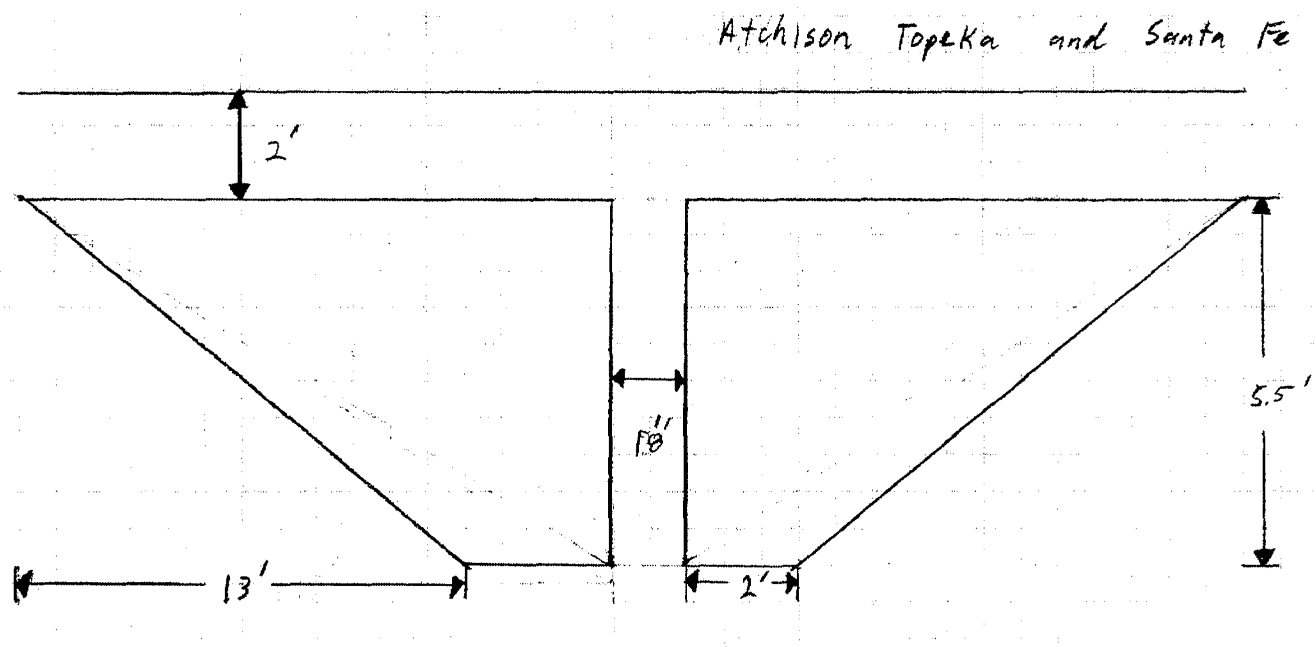
A



Lone Creek and Free Love Ave.



Lone tree Creek and Carrolton rd.



Lone Tree Tributary

TEMPLE CREEK | CROSSING RATINGS

RS 430 JACK TONE RD. DECK = 51.57

Q	ELEV		AREA	AVG DEPTH	STORAGE
300	49.9				
500	50.1	WEIR			
750	52.1		280	1	280
1000	52.7	Lower – model is constrained	355	1.4	500
1250	53.3		450	1.7	770

AT&SF RR

Q	ELEV	AREA	AVG DEPTH	STORAGE
750	63.2			
1000	64.0			
1250	64.3			
1500	65.9			
1750	67.5			

EXCESS FLOWS TO SFSLJ

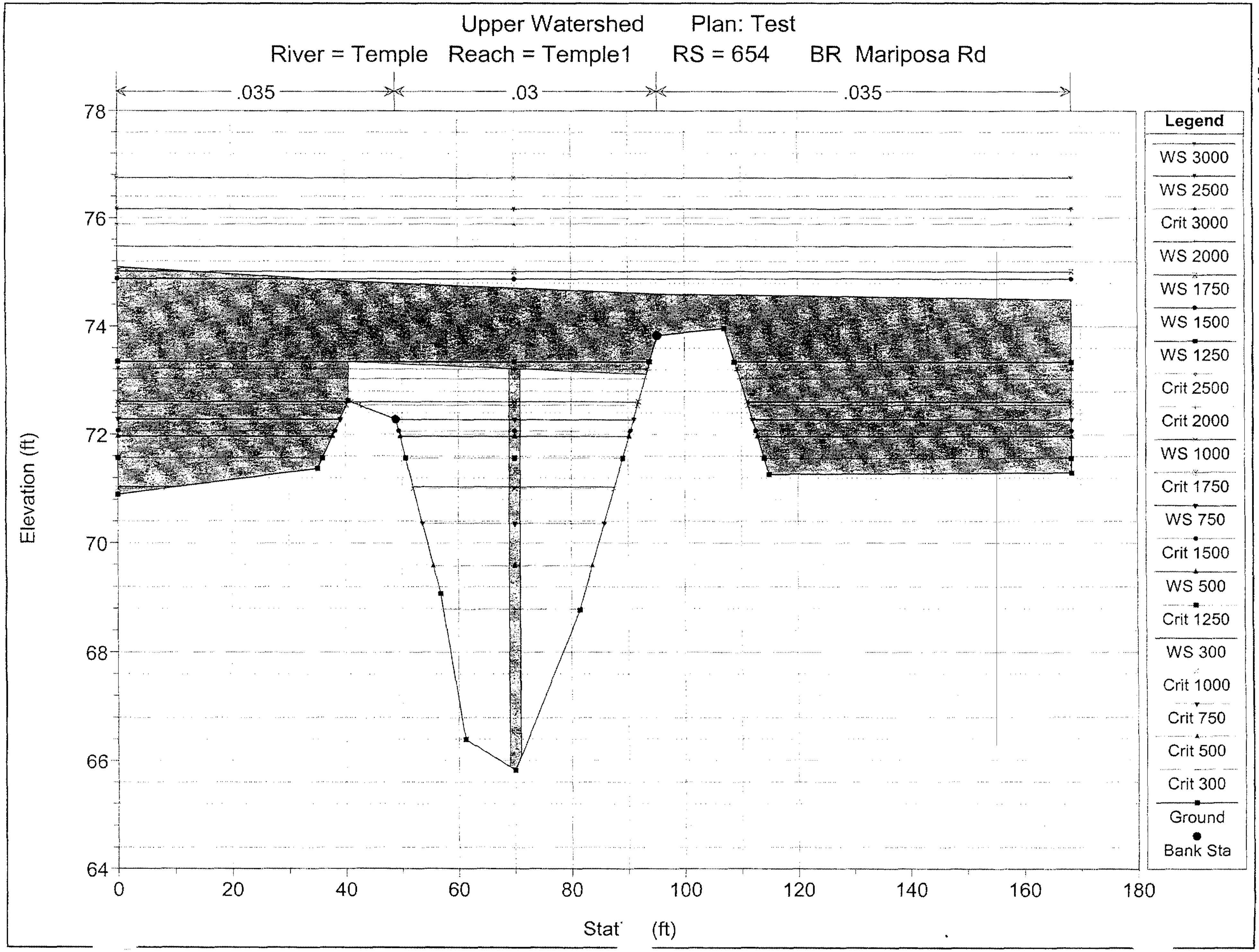
MURPHY RD

Q	ELEV	AREA	AVG DEPTH	STORAGE
500	62.9			
750	64.1	40	1	40
1000	65.65	205	1.5	310
1250	66.4	410	2	820

MARIPOSA RD

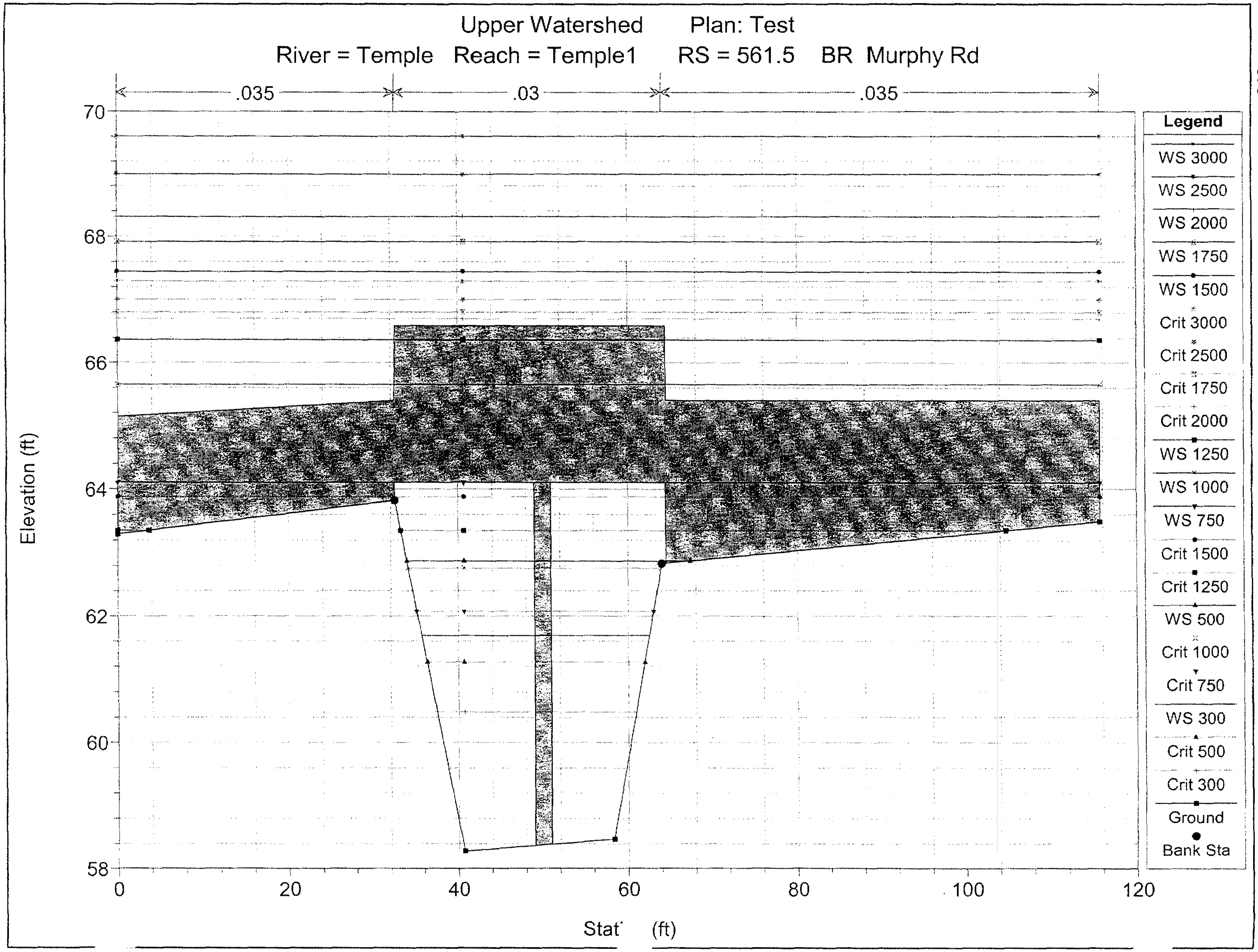
Q	EL	AREA	D	STOR.
1250	72.8			
1500	74.8			
1750	75			
2000	75.3			
2500	75.7			

FLOWS TO SFSLJ



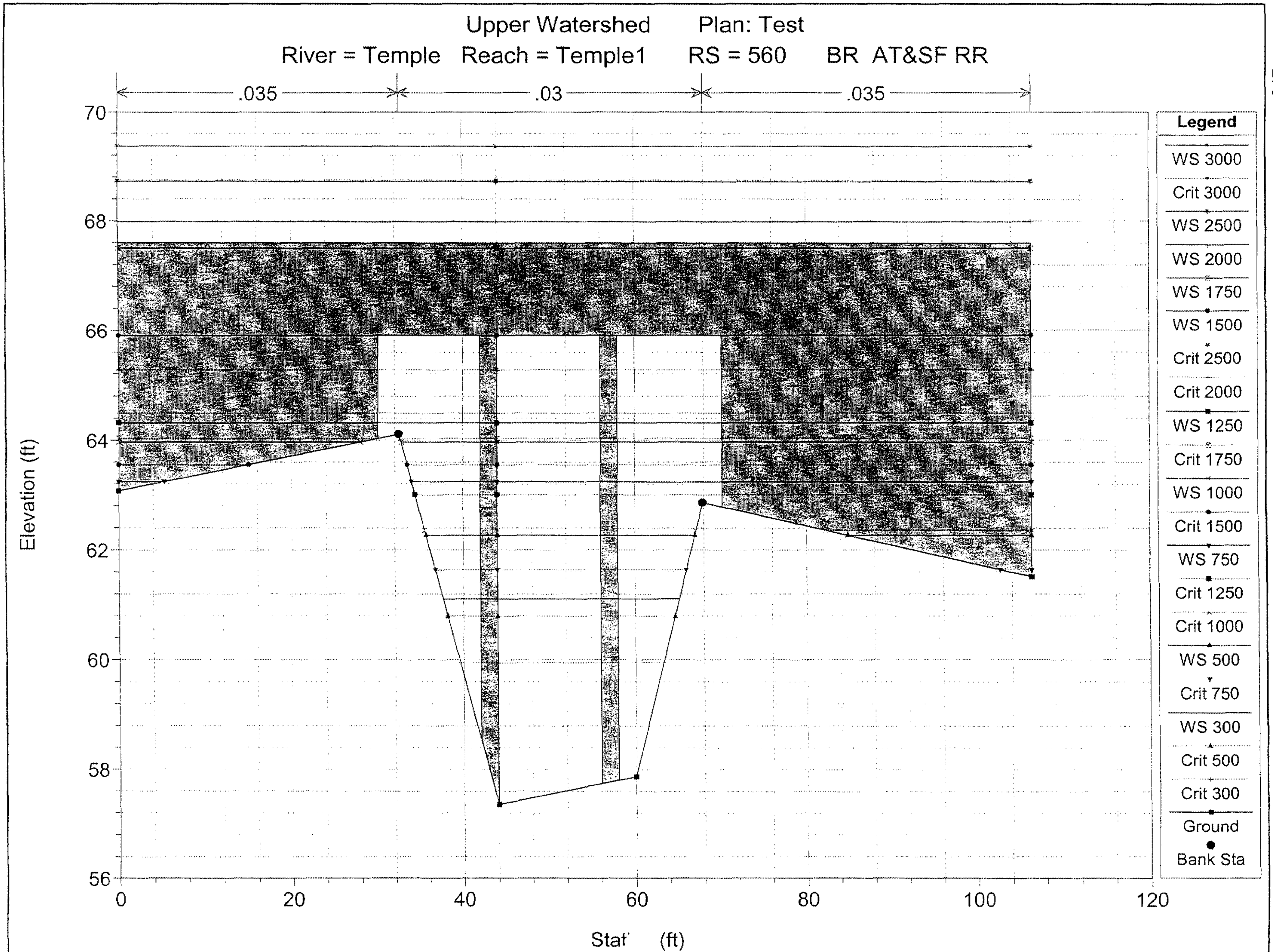
Upper Watershed Plan: Test
River = Temple Reach = Temple1 RS = 654 BR Mariposa Rd
.035
.03
.035
78
76
74
72
70
68
66
64
Elevation (ft)
0
20
40
60
80
100
120
140
160
180
Stat (ft)
Legend
WS 3000
WS 2500
Crit 3000
WS 2000
WS 1750
WS 1500
WS 1250
Crit 2500
Crit 2000
WS 1000
Crit 1750
WS 750
Crit 1500
WS 500
Crit 1250
WS 300
Crit 1000
Crit 750
Crit 500
Crit 300
Ground
Bank Sta

245



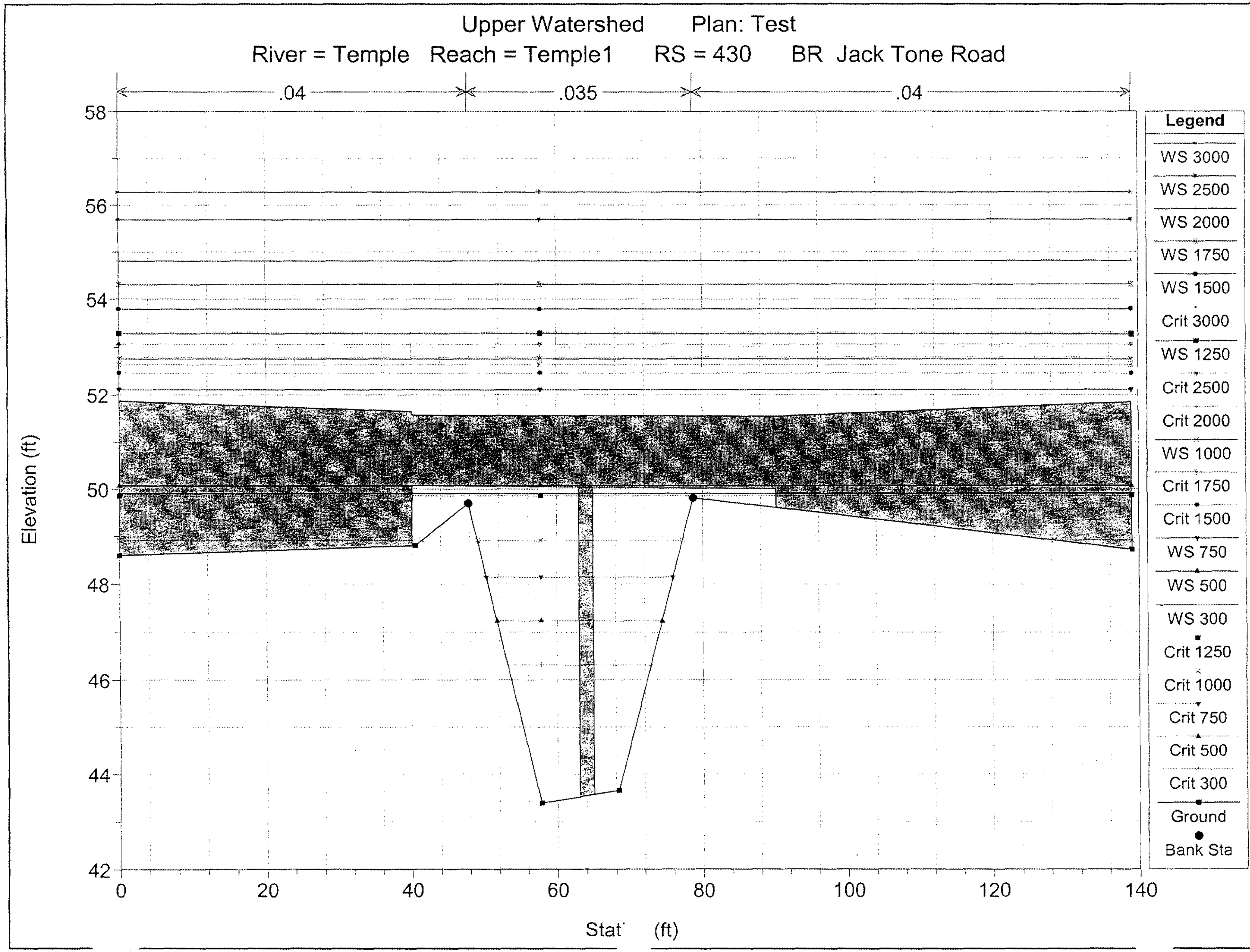
Upper Watershed Plan: Test
River = Temple Reach = Temple1 RS = 561.5 BR Murphy Rd
.035
.03
.035
Elevation (ft)
Stat (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
Crit 3000
Crit 2500
Crit 1750
Crit 2000
WS 1250
WS 1000
WS 750
Crit 1500
Crit 1250
WS 500
Crit 1000
Crit 750
WS 300
Crit 500
Crit 300
Ground
Bank Sta

246

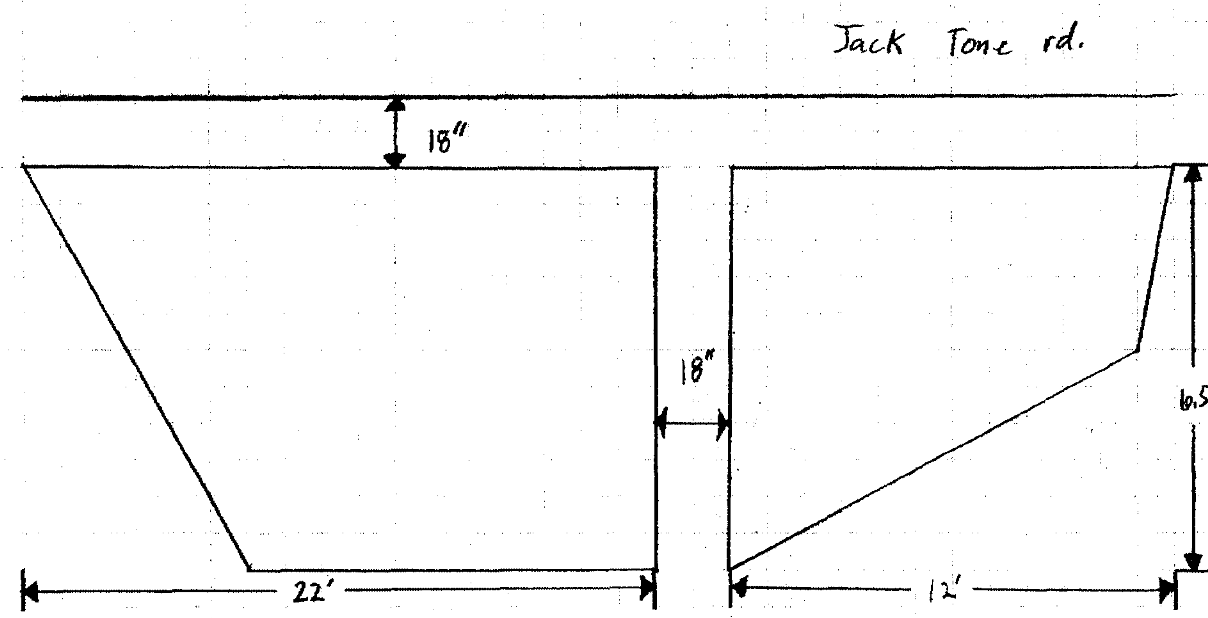


Upper Watershed Plan: Test
River = Temple Reach = Temple1 RS = 560 BR AT&SF RR
.035
.03
.035
Elevation (ft)
Stat (ft)
70
68
66
64
62
60
58
56
0
20
40
60
80
100
120
Legend
WS 3000
Crit 3000
WS 2500
WS 2000
WS 1750
WS 1500
Crit 2500
Crit 2000
WS 1250
Crit 1750
WS 1000
Crit 1500
WS 750
Crit 1250
Crit 1000
WS 500
Crit 750
WS 300
Crit 500
Crit 300
Ground
Bank Sta

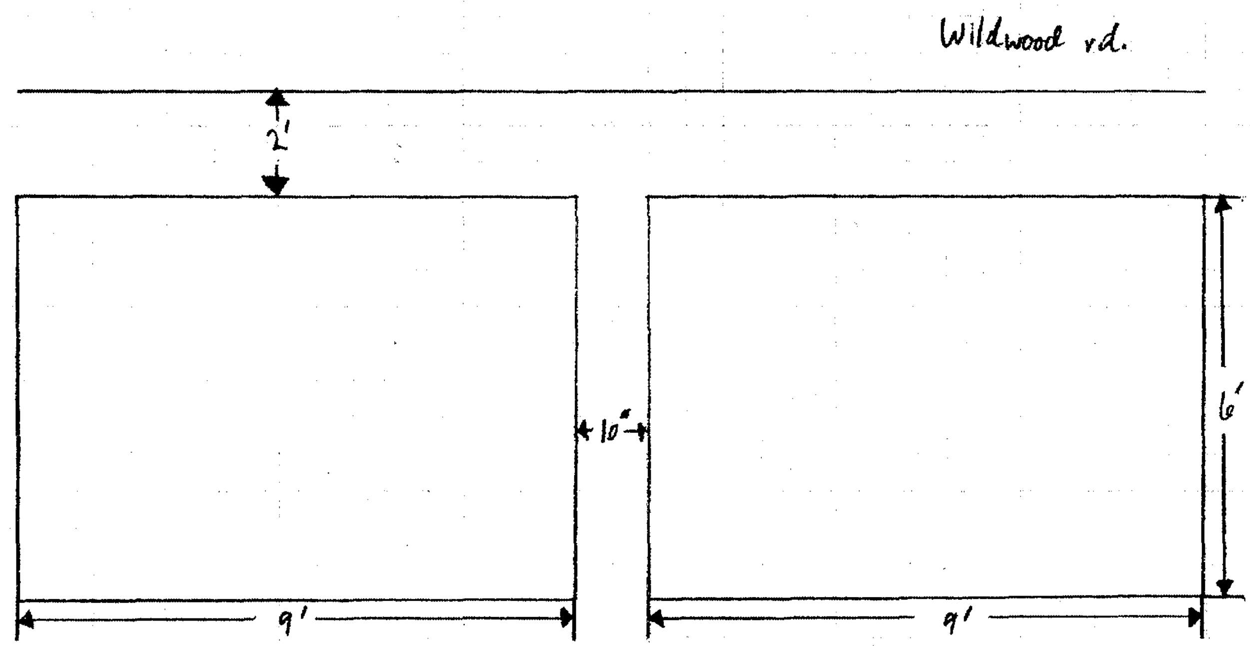
247



Upper Watershed Plan: Test
River = Temple Reach = Temple1 RS = 430 BR Jack Tone Road
.04
.035
.04
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
Crit 3000
WS 1250
Crit 2500
Crit 2000
WS 1000
Crit 1750
Crit 1500
WS 750
WS 500
WS 300
Crit 1250
Crit 1000
Crit 750
Crit 500
Crit 300
Ground
Bank Sta
Elevation (ft)
Stat' (ft)
58
56
54
52
50
48
46
44
42
0
20
40
60
80
100
120
140

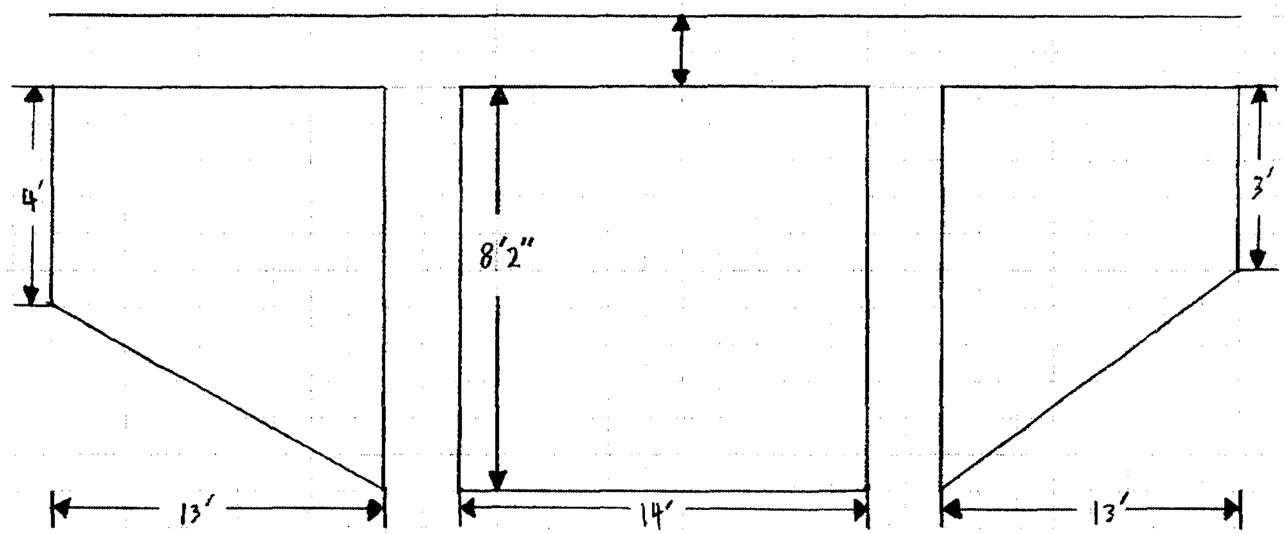


Temple and Jack Tone rd.



Temple and Wildwood rd.

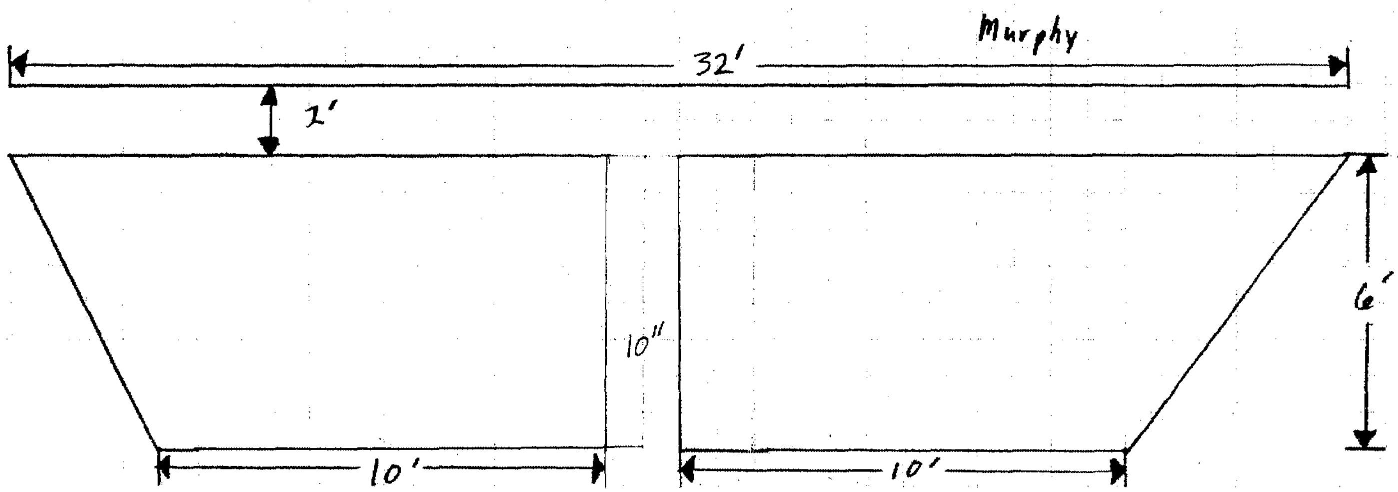
B



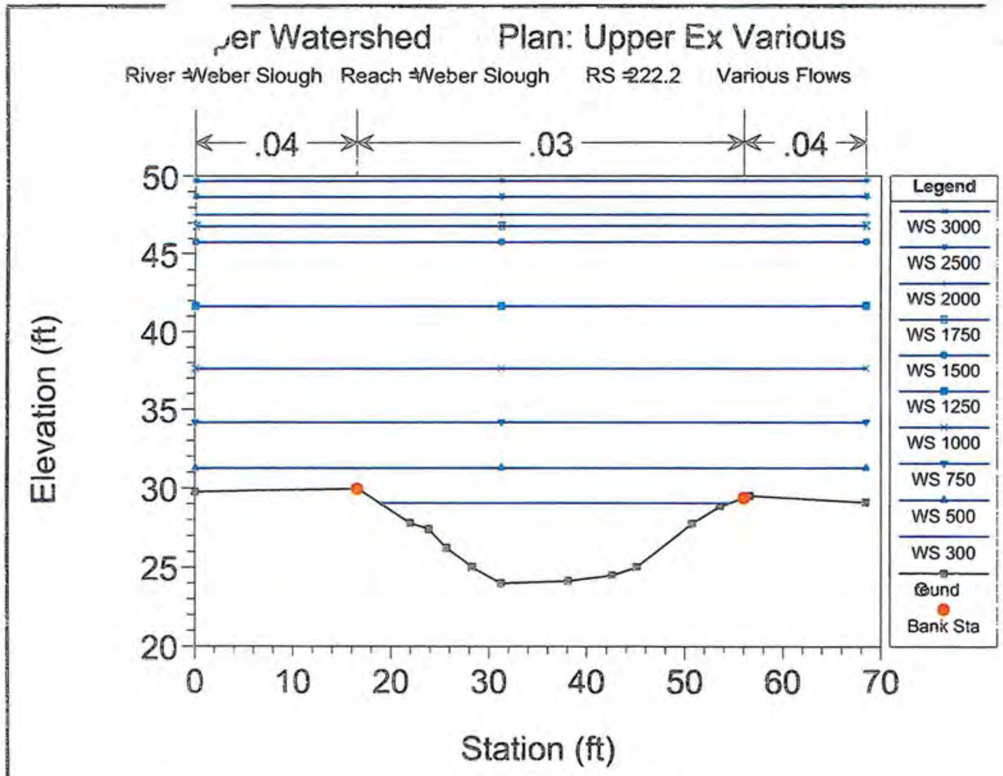
Temple and Atchison Topeka and Santa Fe RR

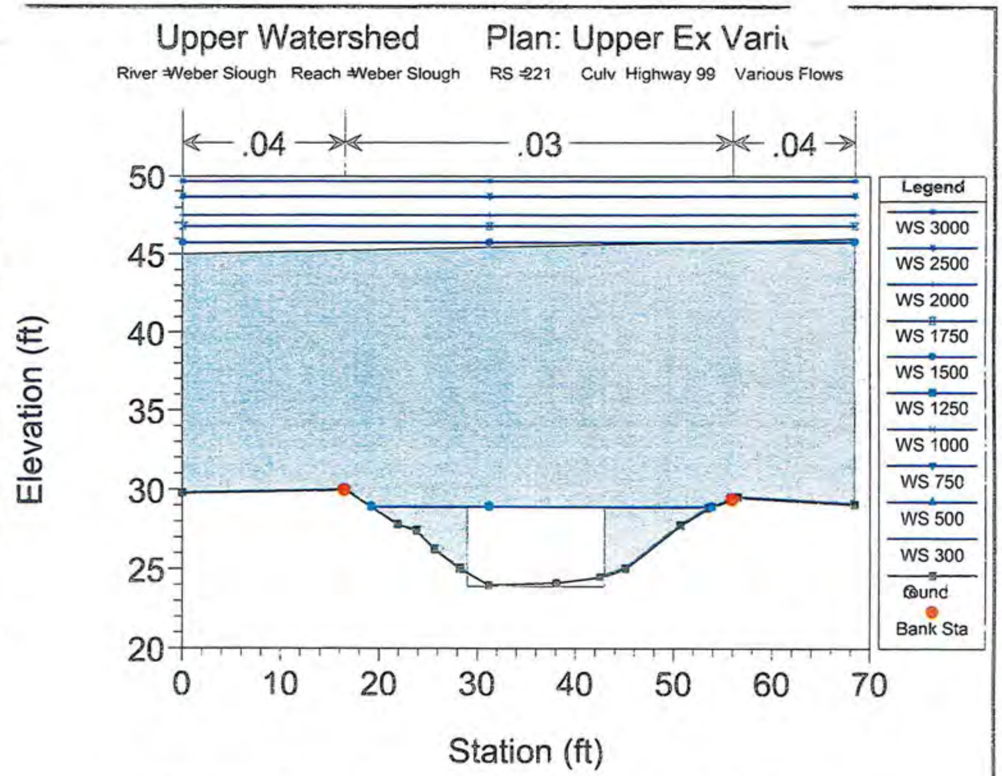
STAEDTLER® No. 937 811E Engineer's Computation Pad

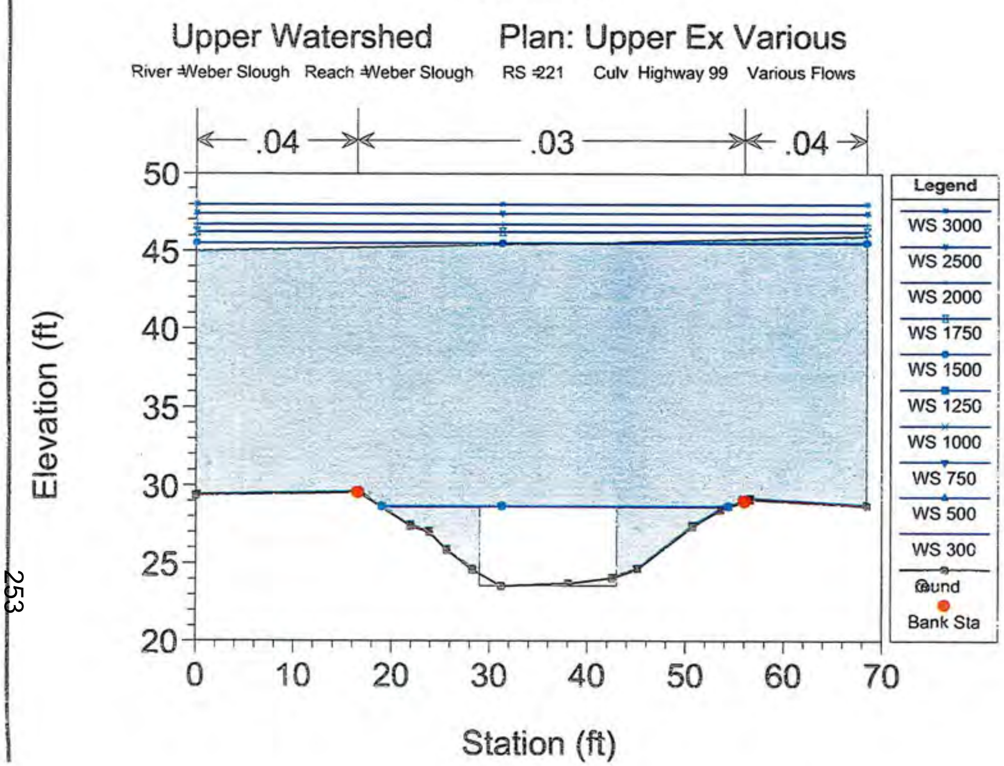
A

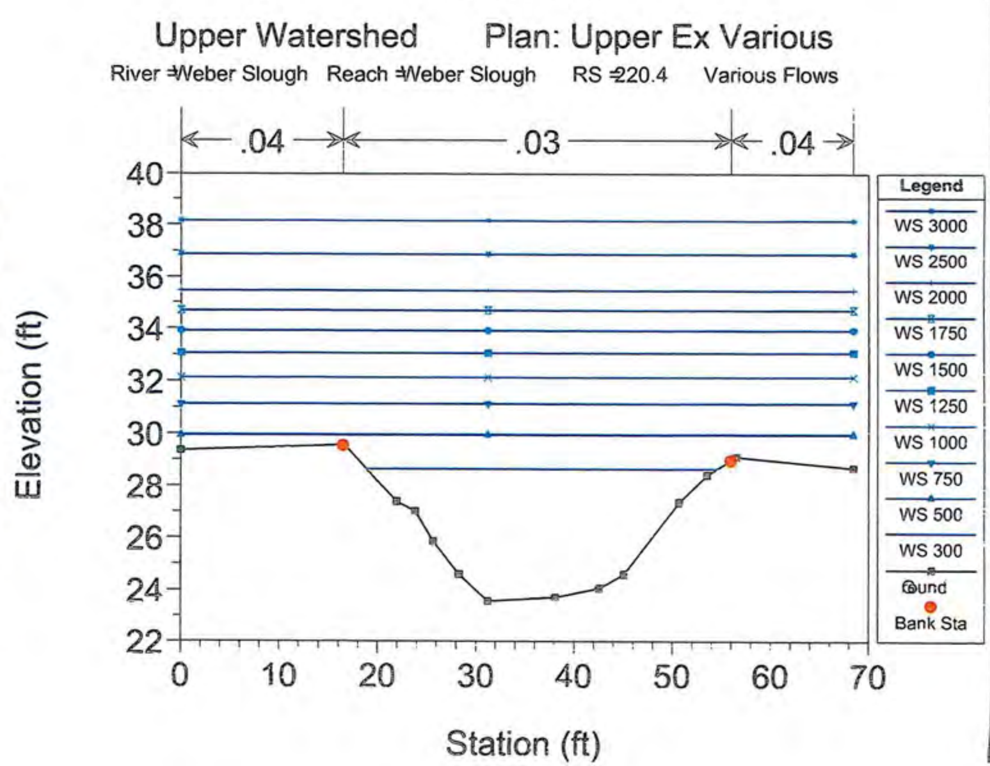


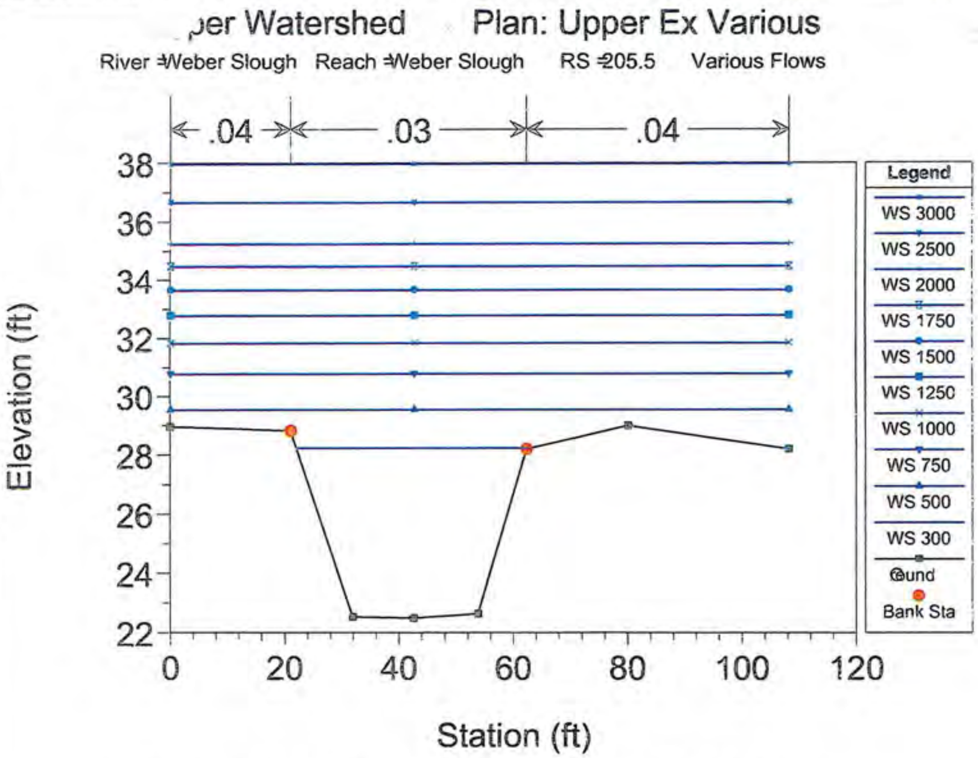
Temple and Murphy rd.

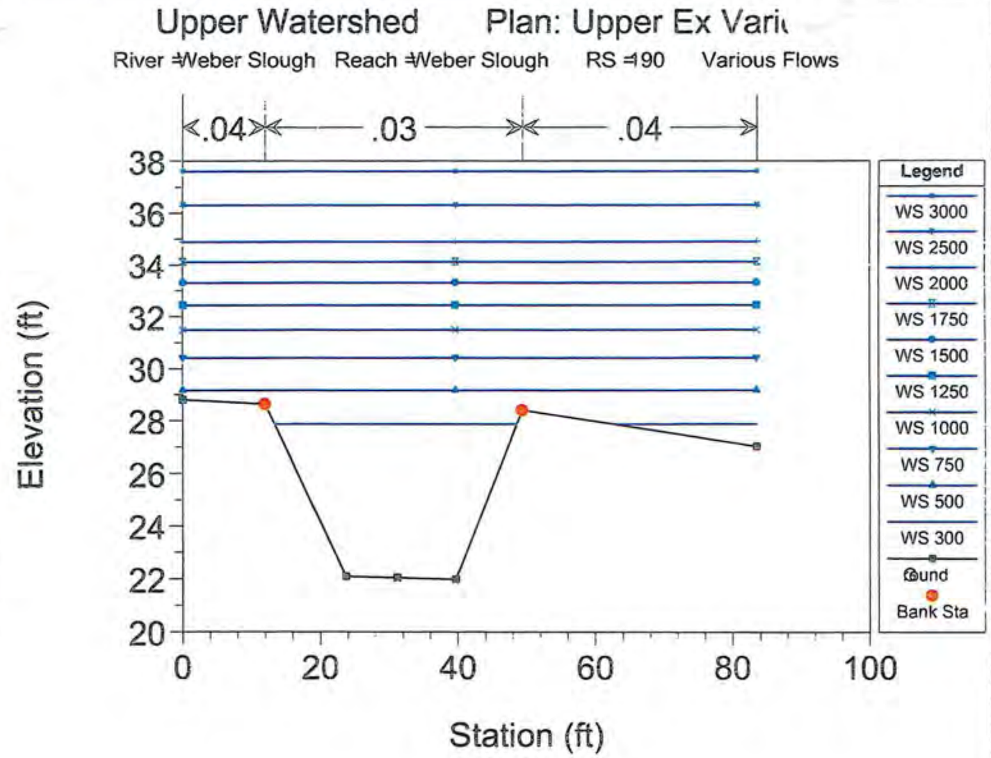

per Watershed Plan: Upper Ex Various
River = Weber Slough Reach = Weber Slough RS = 222.2 Various Flows
.04 .03 .04
Elevation (ft)
Station (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
WS 750
WS 500
WS 300
Ground
Bank Sta

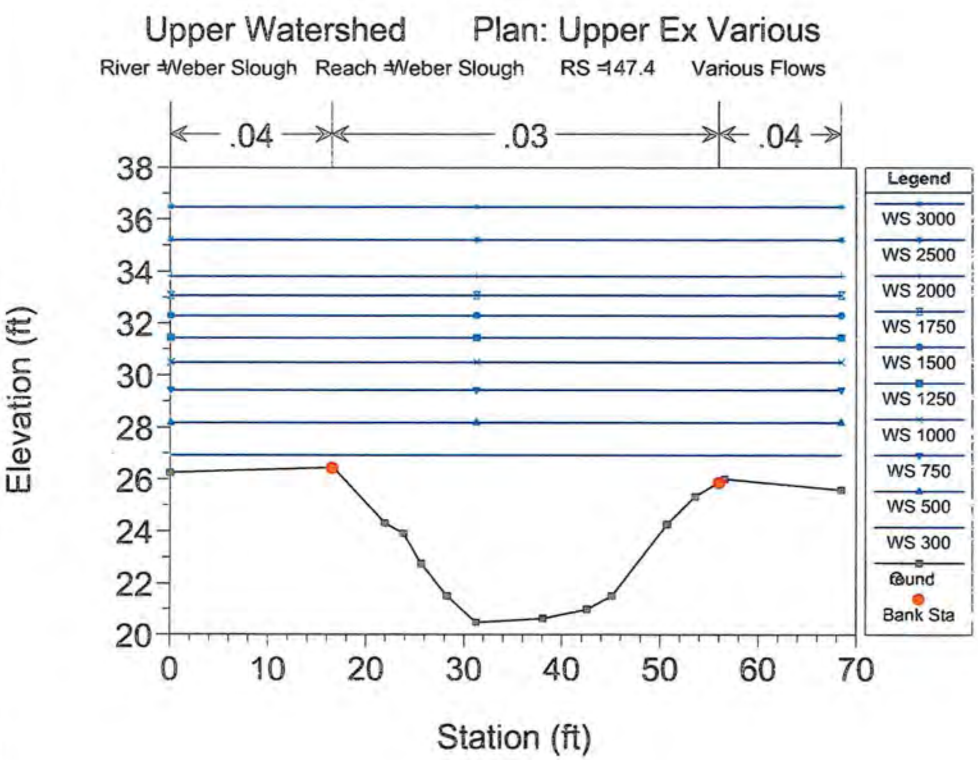

Upper Watershed Plan: Upper Ex Vari
River = Weber Slough Reach = Weber Slough RS = 221 Culv Highway 99 Various Flows
.04 .03 .04
Elevation (ft)
Station (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
WS 750
WS 500
WS 300
Ground
Bank Sta


Upper Watershed Plan: Upper Ex Various
River = Weber Slough Reach = Weber Slough RS = 221 Culv Highway 99 Various Flows
.04 .03 .04
Elevation (ft)
Station (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
WS 750
WS 500
WS 300
Ground
Bank Sta


Upper Watershed Plan: Upper Ex Various
River = Weber Slough Reach = Weber Slough RS = 220.4 Various Flows
.04 .03 .04
Elevation (ft)
Station (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
WS 750
WS 500
WS 300
Ground
Bank Sta


per Watershed Plan: Upper Ex Various
River = Weber Slough Reach = Weber Slough RS = 205.5 Various Flows
.04
.03
.04
Elevation (ft)
Station (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
WS 750
WS 500
WS 300
Ground
Bank Sta


Upper Watershed Plan: Upper Ex Vari
River = Weber Slough Reach = Weber Slough RS = 190 Various Flows
.04
.03
.04
Elevation (ft)
Station (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
WS 750
WS 500
WS 300
Ground
Bank Sta


Upper Watershed Plan: Upper Ex Various
River = Weber Slough Reach = Weber Slough RS = 147.4 Various Flows
.04
.03
.04
Elevation (ft)
Station (ft)
Legend
WS 3000
WS 2500
WS 2000
WS 1750
WS 1500
WS 1250
WS 1000
WS 750
WS 500
WS 300
Ground
Bank Sta

Attachment 6- C. Corpscon Vertical Datum Conversion for French Camp Slough Model Elements

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LSJRFS Hydrology

French Camp Slough HEC-HMS Elevation Conversion

21 December 2010

INPUT	OUTPUT
State Plane, NAD83 0403 - California 3, U.S. Feet Vertical - NGVD29 (Vertcon94), U.S. Feet	State Plane, NAD83 0403 - California 3, U.S. Feet Vertical - NAVD88, U.S. Feet

Farmington (1/4)

	Input	Output
Northing/Y:	2153660	2153660.000
Easting/X:	6436130	6436130.000
Elevation/Z:	0	2.382
Convergence:	-0 15 59.05500	-0 15 59.05500
Scale Factor:	0.999932954	0.999932954
Combined Factor:	0.999937856	0.999937742

Grid Shift (U.S. ft.): X/Easting = 0.0, Y/Northing = 0.0

LT at Austin (2/4)

	Input	Output
Northing/Y:	2134430	2134430.000
Easting/X:	6365780	6365780.000
Elevation/Z:	0	2.326
Convergence:	-0 24 55.42316	-0 24 55.42316
Scale Factor:	0.999930812	0.999930812
Combined Factor:	0.999935810	0.999935699

Grid Shift (U.S. ft.): X/Easting = 0.0, Y/Northing = 0.0

LT at Jack Tone (3/4)

	Input	Output
Northing/Y:	2127750	2127750.000
Easting/X:	6376440	6376440.000
Elevation/Z:	0	2.333
Convergence:	-0 23 33.69650	-0 23 33.69650
Scale Factor:	0.999930291	0.999930291
Combined Factor:	0.999935287	0.999935176

Grid Shift (U.S. ft.): X/Easting = 0.0, Y/Northing = 0.0

SLJ at 99 (4/4)

	Input	Output
Northing/Y:	2140040	2140040.000
Easting/X:	6353920	6353920.000
Elevation/Z:	0	2.290
Convergence:	-0 26 26.28976	-0 26 26.28976
Scale Factor:	0.999931324	0.999931324
Combined Factor:	0.999936325	0.999936216

Grid Shift (U.S. ft.): X/Easting = 0.0, Y/Northing = 0.0

Remark:

Corpscon v6.0.1, U.S. Army Corps of Engineers

Attachment 6- D. French Camp Slough Subbasin Characteristics – Existing Conditions

Subbasin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
ARCH	0.74	0.02	1.60	40	30	0.68	6.25	0.35	VD	1.5	0.021	60	10.8
BACH	43.07	0.04	14.79	1090	150	9.66	63.55	2.87	FH	1.5	0.030	0	
CLAY	0.10	0.02	0.59	12	10	0.34	3.40	0.21	VD	1.5	0.021	40	9.8
DC1	5.60	0.2	6.74	169	80	3.40	13.21	9.66	VU	1.5	0.051	2	
DC10	0.04	0.02	0.47	15	10	0.47	10.62	0.17	VD	1.5	0.021	50	
DC11	0.23	0.02	1.21	15	10	0.55	4.14	0.31	VD	1.5	0.021	40	
DC2	5.91	0.2	7.81	90	46	4.46	5.63	13.32	VU	1.5	0.023	2	
DC3	4.04	0.2	4.40	75	52	2.11	5.23	8.17	VU	1.5	0.024	2	
DC4	3.80	0.2	3.17	53	32	1.31	6.63	5.75	VU	1.5	0.022	2	
DC5	1.68	0.2	2.80	52	40	1.40	4.28	6.12	VU	1.5	0.021	2	
DC6	0.92	0.025	2.34	40	27	0.97	5.55	0.59	VD	1.5	0.021	40	
DC7	0.32	0.02	1.04	30	23	0.49	6.74	0.26	VD	1.5	0.021	50	
DC8	0.62	0.1	1.45	30	20	0.83	6.88	1.78	VU	1.5	0.021	10	
DC9	0.17	0.02	0.62	10	9	0.32	1.60	0.24	VD	1.5	0.021	40	
DCAP	1.79	0.015	2.86	30	10	1.28	6.99	0.41	VD	1.5	0.021	60	114.8
DUCK	28.28	0.04	16.44	315	96	10.04	13.32	4.09	FH	1.5	0.031	0	
FARM	32.48	0.03	12.32	260	150	5.68	8.93	2.39	FH	1.5	0.036	0	
FCS1	1.70	0.1	2.29	27	18	1.04	3.93	2.58	VU	1.5	0.047	5	
FCS2	0.46	0.15	1.27	22	15	0.59	5.50	2.33	VU	1.5	0.086	5	
FCS3	0.26	0.15	1.38	15	11	0.68	2.90	2.87	VU	1.5	0.059	5	
FCS4	0.20	0.025	1.04	15	11	0.52	3.86	0.37	VD	1.5	0.021	40	
FCS5	0.30	0.025	1.15	20	15	0.56	4.36	0.38	VD	1.5	0.021	40	
FCS6	0.38	0.15	1.14	15	10	0.74	4.38	2.55	VU	1.5	0.021	5	
FCS7	0.12	0.15	0.71	11	5	0.41	8.45	1.50	VU	1.5	0.021	5	
GRUPE	0.20	0.015	1.17	15	10	0.73	4.28	0.26	VD	1.5	0.032	60	120.7
GTWY	0.77	0.02	1.42	21	15	0.73	4.22	0.37	VD	1.5	0.021	50	22.3
LJ1	6.30	0.2	4.81	239	96	2.29	29.73	6.27	VU	1.5	0.067	2	
LJ2	1.40	0.2	3.31	100	78	1.80	6.64	6.60	VU	1.5	0.095	2	
LT A1	2.32	0.2	3.40	208	150	1.70	17.06	5.46	VU	1.5	0.021	2	

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Subbasin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
LT A2	3.80	0.2	4.40	214	150	2.21	14.53	6.85	VU	1.5	0.021	2	
LT A3	0.15	0.1	0.70	150	147	0.36	4.28	1.08	VU	1.5	0.111	5	
LT B1	3.20	0.2	4.06	229	157	2.04	17.72	6.20	VU	1.5	0.046	2	
LT B2	3.26	0.2	2.66	155	130	1.42	9.40	5.20	VU	1.5	0.052	2	
LT B3	4.07	0.2	4.19	152	115	1.82	8.84	6.86	VU	1.5	0.035	2	
LT B4	2.75	0.2	4.30	156	115	1.99	9.54	7.07	VU	1.5	0.144	2	
LT B5	2.05	0.2	2.77	115	90	1.36	9.04	5.23	VU	1.5	0.046	2	
LT C1	2.47	0.2	3.48	106	75	1.44	8.90	5.85	VU	1.5	0.080	2	
LT C2	2.66	0.2	2.99	85	60	1.44	8.35	5.59	VU	1.5	0.052	2	
LT C3	0.97	0.2	1.14	50	43	0.83	6.16	3.33	VU	1.5	0.045	2	
LT C4a	1.68	0.2	1.69	48	39	0.76	5.04	3.87	VU	1.5	0.042	2	
LT C4b	1.19	0.2	2.01	40	30	0.97	4.98	4.55	VU	1.5	0.063	2	
LT D1	3.51	0.2	3.71	115	79	1.74	9.70	6.34	VU	1.5	0.027	2	
LT D2	2.25	0.2	4.73	76	50	2.12	5.49	8.34	VU	1.5	0.035	2	
LT E1	8.62	0.2	8.66	115	50	3.48	7.51	11.94	VU	1.5	0.142	2	
LT F1	1.26	0.2	2.24	90	69	1.06	9.36	4.36	VU	1.5	0.021	2	
LT F2	3.28	0.2	4.32	69	44	2.31	5.79	8.24	VU	1.5	0.021	2	
LT G1	0.45	0.2	1.02	31	24	0.44	6.84	2.45	VU	1.5	0.060	2	
NFSLJ1	1.07	0.1	1.99	65	45	1.00	10.06	2.01	VU	1.5	0.021	2	
NFSLJ2	6.78	0.1	5.00	56	18	2.67	7.60	4.37	VU	1.5	0.021	2	
NLJ1	3.33	0.2	4.58	97	76	2.13	4.59	8.55	VU	1.5	0.046	2	
NLJ2	3.22	0.2	4.53	77	50	2.18	5.96	8.17	VU	1.5	0.028	2	
NLJ3	0.75	0.2	1.58	51	40	0.69	6.95	3.44	VU	1.5	0.021	2	
NLJ4	1.19	0.15	2.43	45	30	1.02	6.18	3.60	VU	1.5	0.021	5	
ROCK1	6.19	0.04	4.46	1250	560	2.19	154.63	0.88	FH	1.5	0.025	0	
Rock2	16.01	0.04	7.09	1400	210	3.41	167.78	1.22	FH	1.5	0.023	0	
Rock3	11.88	0.04	10.29	210	150	3.03	5.83	2.54	FH	1.5	0.037	0	
SABC	1.80	0.02	2.40	32	25	0.97	2.92	0.54	VD	1.5	0.021	50	66.8

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Subbasin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
SALT1	2.23	0.04	2.75	2000	1175	1.52	300.41	0.56	FH	1.5	0.022	0	
SALT2	2.39	0.04	2.08	2140	1200	0.85	451.20	0.37	FH	1.5	0.021	0	
SALT3	15.06	0.03	3.84	1240	1075	1.52	42.92	0.69	FH	1.5	0.045	0	
SFSLJ1	0.75	0.2	1.55	58	44	0.76	9.01	3.36	VU	1.5	0.021	2	
SFSLJ2	3.30	0.2	4.70	52	19	2.35	7.03	8.25	VU	1.5	0.021	2	
SFSLJ A1	5.93	0.2	8.66	110	44	4.92	7.63	13.58	VU	1.5	0.044	2	
SLJ	4.45	0.1	6.95	78	49	3.66	4.17	6.25	VU	1.5	0.028	5	
STAGE	0.50	0.015	0.75	30	28	0.29	2.68	0.17	VD	1.5	0.021	60	155.4
TE A1	3.49	0.1	2.25	166	128	1.04	16.86	1.94	VU	1.5	0.021	2	
TE B1	2.72	0.1	4.51	162	105	2.58	12.65	3.76	VU	1.5	0.021	2	
TE B2	2.27	0.1	3.83	128	105	2.09	6.01	3.76	VU	1.5	0.021	2	
TE B3	2.09	0.1	2.31	84	67	1.29	7.36	2.49	VU	1.5	0.021	2	
TE B4	2.83	0.2	4.89	101	59	1.59	8.60	6.95	VU	1.5	0.023	2	
TE C1	2.66	0.2	4.39	71	37	1.80	7.74	7.14	VU	1.5	0.022	2	
TE D1	4.53	0.1	3.18	200	147	1.99	16.66	2.84	VU	1.5	0.027	2	
TE D2	5.34	0.15	8.90	147	99	4.64	5.39	10.75	VU	1.5	0.033	2	
TE D3	2.90	0.15	4.68	107	69	2.50	8.12	6.16	VU	1.5	0.025	2	
TE E1	4.18	0.1	4.77	185	107	2.48	16.34	3.61	VU	1.5	0.021	2	
TE F1	5.13	0.15	6.69	165	85	3.45	11.97	7.40	VU	1.5	0.044	2	
TE F2	3.47	0.15	4.98	128	78	2.65	10.04	6.19	VU	1.5	0.021	2	
TURN	2.33	0.015	2.15	15	8	0.93	3.25	0.37	VD	1.5	0.022	60	116.9
UPLJ1	35.10	0.04	19.98	1860	550	10.61	65.57	3.32	FH	1.5	0.023	0	
UPLJ2	51.62	0.04	20.00	1400	150	11.36	62.49	3.44	FH	1.5	0.030	0	
Web1a	3.72	0.1	4.55	75	45	2.08	6.60	3.94	VU	1.5	0.021	5	
Web1b	1.11	0.1	3.28	45	25	1.70	6.10	3.27	VU	1.5	0.021	5	
Web2a	1.42	0.1	2.27	25	20	1.00	2.20	2.83	VU	1.5	0.021	5	
Web2b	0.89	0.04	1.89	22	12	0.95	5.28	0.87	VD	1.5	0.021	50	50.0
WPIP	0.93	0.02	1.38	19	10	0.40	6.50	0.27	VD	1.5	0.021	50	60.7

Attachment 6- E. French Camp Slough Subbasin Characteristics – Future Conditions

Subbasin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
ARCH	0.74	0.02	1.60	40	30	0.68	6.25	0.35	VD	1.5	0.021	60	10.8
BACH	43.07	0.04	14.79	1090	150	9.66	63.55	2.87	FH	1.5	0.030	0	
CLAY	0.10	0.02	0.59	12	10	0.34	3.40	0.21	VD	1.5	0.021	40	9.8
DC1	5.60	0.2	6.74	169	80	3.40	13.21	9.66	VU	1.5	0.051	2	
DC10	0.04	0.02	0.47	15	10	0.47	10.62	0.17	VD	1.5	0.021	50	9.5
DC11	0.23	0.02	1.21	15	10	0.55	4.14	0.31	VD	1.5	0.021	40	54.5
DC2	5.91	0.2	7.81	90	46	4.46	5.63	13.32	VU	1.5	0.023	2	
DC3	4.04	0.2	4.40	75	52	2.11	5.23	8.17	VU	1.5	0.024	2	
DC4	3.80	0.015	3.17	53	32	1.31	6.63	0.43	VD	1.5	0.022	60	899.8
DC5	1.68	0.015	2.80	52	40	1.40	4.28	0.46	VD	1.5	0.021	60	397.8
DC6	0.92	0.025	2.34	40	27	0.97	5.55	0.59	VD	1.5	0.021	40	217.9
DC7	0.32	0.02	1.04	30	23	0.49	6.74	0.26	VD	1.5	0.021	50	75.8
DC8	0.62	0.015	1.45	30	20	0.83	6.88	0.27	VD	1.5	0.021	60	146.8
DC9	0.17	0.02	0.62	10	9	0.32	1.60	0.24	VD	1.5	0.021	40	40.3
DCAP	1.79	0.015	2.86	30	10	1.28	6.99	0.41	VD	1.5	0.021	60	114.8
DUCK	28.28	0.04	16.44	315	96	10.04	13.32	4.09	FH	1.5	0.031	0	
FARM	32.48	0.03	12.32	260	150	5.68	8.93	2.39	FH	1.5	0.036	0	
FCS1	1.70	0.015	2.29	27	18	1.04	3.93	0.39	VD	1.5	0.047	60	402.6
FCS2	0.46	0.015	1.27	22	15	0.59	5.50	0.23	VD	1.5	0.086	60	108.9
FCS3	0.26	0.015	1.38	15	11	0.68	2.90	0.29	VD	1.5	0.059	60	61.6
FCS4	0.20	0.025	1.04	15	11	0.52	3.86	0.37	VD	1.5	0.021	40	47.4
FCS5	0.30	0.025	1.15	20	15	0.56	4.36	0.38	VD	1.5	0.021	40	71.0
FCS6	0.38	0.015	1.14	15	10	0.74	4.38	0.26	VD	1.5	0.021	60	90.0
FCS7	0.12	0.015	0.71	11	5	0.41	8.45	0.15	VD	1.5	0.021	60	28.4
GRUPE	0.20	0.015	1.17	15	10	0.73	4.28	0.26	VD	1.5	0.032	60	120.7
GTWY	0.77	0.02	1.42	21	15	0.73	4.22	0.37	VD	1.5	0.021	50	22.3
LJ1	6.30	0.2	4.81	239	96	2.29	29.73	6.27	VU	1.5	0.067	2	
LJ2	1.40	0.2	3.31	100	78	1.80	6.64	6.60	VU	1.5	0.095	2	
LT A1	2.32	0.2	3.40	208	150	1.70	17.06	5.46	VU	1.5	0.021	2	

Subbasin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
LT A2	3.80	0.2	4.40	214	150	2.21	14.53	6.85	VU	1.5	0.021	2	
LT A3	0.15	0.1	0.70	150	147	0.36	4.28	1.08	VU	1.5	0.111	5	
LT B1	3.20	0.2	4.06	229	157	2.04	17.72	6.20	VU	1.5	0.046	2	
LT B2	3.26	0.2	2.66	155	130	1.42	9.40	5.20	VU	1.5	0.052	2	
LT B3	4.07	0.2	4.19	152	115	1.82	8.84	6.86	VU	1.5	0.035	2	
LT B4	2.75	0.2	4.30	156	115	1.99	9.54	7.07	VU	1.5	0.144	2	
LT B5	2.05	0.2	2.77	115	90	1.36	9.04	5.23	VU	1.5	0.046	2	
LT C1	2.47	0.2	3.48	106	75	1.44	8.90	5.85	VU	1.5	0.080	2	
LT C2	2.66	0.2	2.99	85	60	1.44	8.35	5.59	VU	1.5	0.052	2	
LT C3	0.97	0.2	1.14	50	43	0.83	6.16	3.33	VU	1.5	0.045	2	
LT C4a	1.68	0.2	1.69	48	39	0.76	5.04	3.87	VU	1.5	0.042	2	
LT C4b	1.19	0.015	2.01	40	30	0.97	4.98	0.34	VD	1.5	0.063	60	281.8
LT D1	3.51	0.2	3.71	115	79	1.74	9.70	6.34	VU	1.5	0.027	2	
LT D2	2.25	0.2	4.73	76	50	2.12	5.49	8.34	VU	1.5	0.035	2	
LT E1	8.62	0.2	8.66	115	50	3.48	7.51	11.94	VU	1.5	0.142	2	
LT F1	1.26	0.2	2.24	90	69	1.06	9.36	4.36	VU	1.5	0.021	2	
LT F2	3.28	0.2	4.32	69	44	2.31	5.79	8.24	VU	1.5	0.021	2	
LT G1	0.45	0.015	1.02	31	24	0.44	6.84	0.18	VD	1.5	0.060	60	106.6
NFSLJ1	1.07	0.1	1.99	65	45	1.00	10.06	2.01	VU	1.5	0.021	2	
NFSLJ2	6.78	0.015	5.00	56	18	2.67	7.60	0.66	VD	1.5	0.021	60	1605.5
NLJ1	3.33	0.2	4.58	97	76	2.13	4.59	8.55	VU	1.5	0.046	2	
NLJ2	3.22	0.2	4.53	77	50	2.18	5.96	8.17	VU	1.5	0.028	2	
NLJ3	0.75	0.015	1.58	51	40	0.69	6.95	0.26	VD	1.5	0.021	60	177.6
NLJ4	1.19	0.015	2.43	45	30	1.02	6.18	0.36	VD	1.5	0.021	60	281.8
ROCK1	6.19	0.04	4.46	1250	560	2.19	154.63	0.88	FH	1.5	0.025	0	
Rock2	16.01	0.04	7.09	1400	210	3.41	167.78	1.22	FH	1.5	0.023	0	
Rock3	11.88	0.04	10.29	210	150	3.03	5.83	2.54	FH	1.5	0.037	0	
SABC	1.80	0.02	2.40	32	25	0.97	2.92	0.54	VD	1.5	0.021	50	66.8
SALT1	2.23	0.04	2.75	2000	1175	1.52	300.41	0.56	FH	1.5	0.022	0	

Subbasin	Area	Basin 'n'	Watercourse Length	Upstream Elevation	Downstream Elevation	Length from Centroid	Watercourse Slope	Lag Time	S-Graph	Initial Loss	Constant Loss Rate	Impervious %	Pump Station Capacity
		n	L			Lc	S	Lg					
	[Sq. Mi.]		[miles]	[feet]	[feet]	[miles]	[ft/mile]	[hrs]		[inches]	[in/hour]	[%]	[cfs]
SALT2	2.39	0.04	2.08	2140	1200	0.85	451.20	0.37	FH	1.5	0.021	0	
SALT3	15.06	0.03	3.84	1240	1075	1.52	42.92	0.69	FH	1.5	0.045	0	
SFSLJ1	0.75	0.2	1.55	58	44	0.76	9.01	3.36	VU	1.5	0.021	2	
SFSLJ2	3.30	0.015	4.70	52	19	2.35	7.03	0.62	VD	1.5	0.021	60	781.4
SFSLJ A1	5.93	0.2	8.66	110	44	4.92	7.63	13.58	VU	1.5	0.044	2	
SLJ	4.45	0.1	6.95	78	49	3.66	4.17	6.25	VU	1.5	0.028	5	
STAGE	0.50	0.015	0.75	30	28	0.29	2.68	0.17	VD	1.5	0.021	60	155.4
TE A1	3.49	0.1	2.25	166	128	1.04	16.86	1.94	VU	1.5	0.021	2	
TE B1	2.72	0.1	4.51	162	105	2.58	12.65	3.76	VU	1.5	0.021	2	
TE B2	2.27	0.1	3.83	128	105	2.09	6.01	3.76	VU	1.5	0.021	2	
TE B3	2.09	0.1	2.31	84	67	1.29	7.36	2.49	VU	1.5	0.021	2	
TE B4	2.83	0.2	4.89	101	59	1.59	8.60	6.95	VU	1.5	0.023	2	
TE C1	2.66	0.2	4.39	71	37	1.80	7.74	7.14	VU	1.5	0.022	2	
TE D1	4.53	0.1	3.18	200	147	1.99	16.66	2.84	VU	1.5	0.027	2	
TE D2	5.34	0.15	8.90	147	99	4.64	5.39	10.75	VU	1.5	0.033	2	
TE D3	2.90	0.15	4.68	107	69	2.50	8.12	6.16	VU	1.5	0.025	2	
TE E1	4.18	0.1	4.77	185	107	2.48	16.34	3.61	VU	1.5	0.021	2	
TE F1	5.13	0.15	6.69	165	85	3.45	11.97	7.40	VU	1.5	0.044	2	
TE F2	3.47	0.15	4.98	128	78	2.65	10.04	6.19	VU	1.5	0.021	2	
TURN	2.33	0.015	2.15	15	8	0.93	3.25	0.37	VD	1.5	0.022	60	116.9
UPLJ1	35.10	0.04	19.98	1860	550	10.61	65.57	3.32	FH	1.5	0.023	0	
UPLJ2	51.62	0.04	20.00	1400	150	11.36	62.49	3.44	FH	1.5	0.030	0	
Web1a	3.72	0.1	4.55	75	45	2.08	6.60	3.94	VU	1.5	0.021	5	
Web1b	1.11	0.015	3.28	45	25	1.70	6.10	0.49	VD	1.5	0.021	60	262.8
Web2a	1.42	0.015	2.27	25	20	1.00	2.20	0.42	VD	1.5	0.021	60	336.3
Web2b	0.89	0.04	1.89	22	12	0.95	5.28	0.87	VD	1.5	0.021	50	50.0
WPIP	0.93	0.02	1.38	19	10	0.40	6.50	0.27	VD	1.5	0.021	50	60.7

Attachment 6- F. French Camp Slough Subbasin Soil Groups and Loss Rates

Soil Group:	A (0.35 in/hr)	B (0.2 in/hr)	C (0.1 in/hr)	D (0.025 in/hr)	Composite Loss Rate	Adjusted Loss Rate (Adjustment Factor = 0.85)
Subbasin	[sq. mi.]	[sq. mi.]	[sq. mi.]	[sq. mi.]	[in/hr]	[in/hr]
ARCH	0.00	0.00	0.00	0.74	0.025	0.021
BACH	0.00	1.03	3.54	38.69	0.035	0.030
CLAY	0.00	0.00	0.00	0.10	0.025	0.021
DC1	0.00	0.02	2.56	3.01	0.060	0.051
DC10	0.00	0.00	0.00	0.04	0.025	0.021
DC11	0.00	0.00	0.00	0.23	0.025	0.021
DC2	0.00	0.00	0.19	5.71	0.027	0.023
DC3	0.00	0.06	0.07	3.92	0.029	0.024
DC4	0.00	0.00	0.02	3.78	0.025	0.022
DC5	0.00	0.00	0.01	1.67	0.025	0.021
DC6	0.00	0.00	0.00	0.92	0.025	0.021
DC7	0.00	0.00	0.00	0.32	0.025	0.021
DC8	0.00	0.00	0.00	0.62	0.025	0.021
DC9	0.00	0.00	0.00	0.16	0.025	0.021
DCAP	0.00	0.00	0.00	1.79	0.025	0.021
DUCK	0.00	1.58	0.50	26.15	0.036	0.031
FARM	0.00	1.57	3.30	24.54	0.043	0.036
FCS1	0.00	0.29	0.01	1.39	0.056	0.047
FCS2	0.00	0.20	0.00	0.26	0.101	0.086
FCS3	0.00	0.06	0.00	0.19	0.070	0.059
FCS4	0.00	0.00	0.00	0.18	0.025	0.021
FCS5	0.00	0.00	0.00	0.30	0.025	0.021
FCS6	0.00	0.00	0.00	0.34	0.025	0.021
FCS7	0.00	0.00	0.00	0.01	0.025	0.021
GRUPE	0.00	0.01	0.00	0.17	0.037	0.032
GTWY	0.00	0.00	0.00	0.77	0.025	0.021
LJ1	0.00	0.88	2.47	2.91	0.079	0.067
LJ2	0.00	0.16	1.24	0.00	0.111	0.095
LT A1	0.00	0.00	0.00	2.44	0.025	0.021
LT A2	0.00	0.00	0.00	4.00	0.025	0.021
LT A3	0.00	0.10	0.00	0.06	0.131	0.111
LT B1	0.00	0.00	1.36	2.20	0.054	0.046
LT B2	0.00	0.51	0.48	2.44	0.062	0.052
LT B3	0.05	0.11	0.38	3.53	0.041	0.035
LT B4	0.53	0.99	0.65	0.57	0.169	0.144
LT B5	0.04	0.32	0.03	2.03	0.054	0.046
LT C1	0.00	1.12	0.06	1.71	0.095	0.080
LT C2	0.02	0.31	0.67	2.13	0.061	0.052
LT C3	0.00	0.04	0.29	0.69	0.053	0.045
LT C4a	0.09	0.01	0.19	1.58	0.049	0.042
LT C4b	0.03	0.23	0.38	0.95	0.074	0.063

Soil Group:	A (0.35 in/hr)	B (0.2 in/hr)	C (0.1 in/hr)	D (0.025 in/hr)	Composite Loss Rate	Adjusted Loss Rate (Adjustment Factor = 0.85)
Subbasin	[sq. mi.]	[sq. mi.]	[sq. mi.]	[sq. mi.]	[in/hr]	[in/hr]
LT D1	0.00	0.00	0.35	3.55	0.032	0.027
LT D2	0.00	0.00	0.58	2.07	0.041	0.035
LT E1	0.84	3.83	3.62	0.24	0.167	0.142
LT F1	0.00	0.00	0.00	1.26	0.025	0.021
LT F2	0.00	0.00	0.00	3.26	0.025	0.021
LT G1	0.00	0.09	0.06	0.30	0.070	0.060
NFSLJ1	0.00	0.00	0.00	1.07	0.025	0.021
NFSLJ2	0.00	0.00	0.00	6.55	0.025	0.021
NLJ1	0.00	0.01	1.28	2.05	0.054	0.046
NLJ2	0.00	0.15	0.00	3.06	0.033	0.028
NLJ3	0.00	0.00	0.00	0.75	0.025	0.021
NLJ4	0.00	0.00	0.00	1.19	0.025	0.021
ROCK1	0.00	0.00	0.39	5.78	0.030	0.025
Rock2	0.00	0.07	0.39	15.62	0.028	0.023
Rock3	0.00	0.89	0.79	10.20	0.043	0.037
SABC	0.00	0.00	0.00	1.80	0.025	0.021
SALT1	0.00	0.00	0.03	2.20	0.026	0.022
SALT2	0.00	0.00	0.00	2.39	0.025	0.021
SALT3	0.00	0.23	4.46	8.74	0.053	0.045
SFSLJ1	0.00	0.00	0.00	0.75	0.025	0.021
SFSLJ2	0.00	0.00	0.00	3.23	0.025	0.021
SFSLJ A1	0.00	0.20	1.67	4.06	0.052	0.044
SLJ	0.00	0.00	0.48	3.97	0.033	0.028
STAGE	0.00	0.00	0.00	0.50	0.025	0.021
TE A1	0.00	0.00	0.00	3.64	0.025	0.021
TE B1	0.00	0.00	0.00	3.62	0.025	0.021
TE B2	0.00	0.00	0.00	3.03	0.025	0.021
TE B3	0.00	0.00	0.00	2.09	0.025	0.021
TE B4	0.00	0.00	0.11	3.67	0.027	0.023
TE C1	0.00	0.00	0.03	3.29	0.026	0.022
TE D1	0.00	0.19	0.00	4.34	0.032	0.027
TE D2	0.00	0.56	0.01	6.55	0.039	0.033
TE D3	0.00	0.00	0.22	3.40	0.030	0.025
TE E1	0.00	0.00	0.00	4.16	0.025	0.021
TE F1	0.00	0.00	2.11	3.93	0.051	0.044
TE F2	0.00	0.00	0.00	4.08	0.025	0.021
TURN	0.00	0.00	0.04	2.28	0.026	0.022
UPLJ1	0.00	0.00	1.07	34.04	0.027	0.023
UPLJ2	0.00	0.73	5.25	46.11	0.035	0.030
Web1a	0.00	0.00	0.00	3.72	0.025	0.021
Web1b	0.00	0.00	0.00	1.11	0.025	0.021
Web2a	0.00	0.00	0.00	1.42	0.025	0.021

Soil Group:	A (0.35 in/hr)	B (0.2 in/hr)	C (0.1 in/hr)	D (0.025 in/hr)	Composite Loss Rate	Adjusted Loss Rate (Adjustment Factor = 0.85)
Subbasin	[sq. mi.]	[sq. mi.]	[sq. mi.]	[sq. mi.]	[in/hr]	[in/hr]
Web2b	0.00	0.00	0.00	0.89	0.025	0.021
WPAC	0.00	0.00	0.00	0.93	0.025	0.021

Attachment 6- G. French Camp Slough Depth-Duration-Frequency Tables

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SCK

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.171	0.204	0.249	0.284	0.319	0.356	0.408
10 min	0.187	0.245	0.292	0.357	0.406	0.458	0.511	0.584
15 min	0.227	0.296	0.353	0.431	0.491	0.553	0.618	0.707
30 min	0.311	0.407	0.486	0.593	0.675	0.760	0.849	0.971
60 min	0.430	0.563	0.671	0.819	0.934	1.051	1.174	1.343
3 hour	0.674	0.837	0.977	1.180	1.346	1.524	1.718	2.000
6 hour	0.893	1.097	1.274	1.529	1.740	1.967	2.215	2.576
12 hour	1.175	1.463	1.703	2.041	2.309	2.589	2.886	3.302
24 hour	1.621	2.050	2.397	2.868	3.229	3.595	3.972	4.482
48 hour	2.014	2.529	2.945	3.503	3.929	4.358	4.797	5.388
72 hour	2.273	2.839	3.295	3.908	4.375	4.847	5.329	5.979
96 hour	2.490	3.101	3.593	4.253	4.754	5.258	5.773	6.465

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
10 min	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
15 min	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
30 min	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
60 min	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
3 hour	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
6 hour	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
12 hour	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
24 hour	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
48 hour	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
72 hour	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854
96 hour	0.854	0.854	0.854	0.854	0.854	0.854	0.854	0.854

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.112	0.146	0.174	0.213	0.243	0.272	0.304	0.348
10 min	0.160	0.209	0.249	0.305	0.347	0.391	0.436	0.499
15 min	0.194	0.253	0.301	0.368	0.419	0.472	0.528	0.604
30 min	0.266	0.348	0.415	0.506	0.576	0.649	0.725	0.829
60 min	0.367	0.481	0.573	0.699	0.798	0.898	1.003	1.147
3 hour	0.576	0.715	0.834	1.008	1.149	1.301	1.467	1.708
6 hour	0.763	0.937	1.088	1.306	1.486	1.680	1.892	2.200
12 hour	1.003	1.249	1.454	1.743	1.972	2.211	2.465	2.820
24 hour	1.384	1.751	2.047	2.449	2.758	3.070	3.392	3.828
48 hour	1.720	2.160	2.515	2.992	3.355	3.722	4.097	4.601
72 hour	1.941	2.425	2.814	3.337	3.736	4.139	4.551	5.106
96 hour	2.126	2.648	3.068	3.632	4.060	4.490	4.930	5.521

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SCK

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.171	0.204	0.249	0.284	0.319	0.356	0.408
10 min	0.187	0.245	0.292	0.357	0.406	0.458	0.511	0.584
15 min	0.227	0.296	0.353	0.431	0.491	0.553	0.618	0.707
30 min	0.311	0.407	0.486	0.593	0.675	0.760	0.849	0.971
60 min	0.430	0.563	0.671	0.819	0.934	1.051	1.174	1.343
3 hour	0.674	0.837	0.977	1.180	1.346	1.524	1.718	2.000
6 hour	0.893	1.097	1.274	1.529	1.740	1.967	2.215	2.576
12 hour	1.175	1.463	1.703	2.041	2.309	2.589	2.886	3.302
24 hour	1.621	2.050	2.397	2.868	3.229	3.595	3.972	4.482
48 hour	2.014	2.529	2.945	3.503	3.929	4.358	4.797	5.388
72 hour	2.273	2.839	3.295	3.908	4.375	4.847	5.329	5.979
96 hour	2.490	3.101	3.593	4.253	4.754	5.258	5.773	6.465

Farmington Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.605	0.606	0.602	0.604	0.600	0.603	0.603	0.602
10 min	0.609	0.608	0.603	0.605	0.603	0.602	0.602	0.600
15 min	0.608	0.607	0.603	0.604	0.602	0.602	0.602	0.602
30 min	0.605	0.605	0.604	0.601	0.602	0.601	0.600	0.600
60 min	0.609	0.606	0.606	0.605	0.604	0.604	0.603	0.602
3 hour	0.604	0.601	0.600	0.598	0.598	0.597	0.598	0.598
6 hour	0.601	0.598	0.595	0.595	0.594	0.594	0.594	0.595
12 hour	0.600	0.595	0.593	0.591	0.591	0.590	0.590	0.590
24 hour	0.599	0.596	0.595	0.593	0.592	0.592	0.591	0.590
48 hour	0.598	0.595	0.594	0.593	0.593	0.593	0.593	0.593
72 hour	0.595	0.593	0.592	0.591	0.591	0.591	0.592	0.593
96 hour	0.595	0.593	0.592	0.591	0.592	0.592	0.593	0.594

Farmington Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.079	0.104	0.123	0.150	0.170	0.192	0.215	0.246
10 min	0.114	0.149	0.176	0.216	0.245	0.276	0.308	0.350
15 min	0.138	0.180	0.213	0.260	0.296	0.333	0.372	0.426
30 min	0.188	0.246	0.294	0.356	0.406	0.457	0.509	0.583
60 min	0.262	0.341	0.407	0.495	0.564	0.635	0.708	0.808
3 hour	0.407	0.503	0.586	0.706	0.805	0.910	1.027	1.196
6 hour	0.537	0.656	0.758	0.910	1.034	1.168	1.316	1.533
12 hour	0.705	0.870	1.010	1.206	1.365	1.528	1.703	1.948
24 hour	0.971	1.222	1.426	1.701	1.912	2.128	2.347	2.644
48 hour	1.204	1.505	1.749	2.077	2.330	2.584	2.845	3.195
72 hour	1.352	1.684	1.951	2.310	2.586	2.865	3.155	3.546
96 hour	1.482	1.839	2.127	2.514	2.814	3.113	3.423	3.840

FRENCH CAMP SLOUGH

NOAA14 Precipitation Frequency Depths

Rainfall Zone: SCK

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.171	0.204	0.249	0.284	0.319	0.356	0.408
10 min	0.187	0.245	0.292	0.357	0.406	0.458	0.511	0.584
15 min	0.227	0.296	0.353	0.431	0.491	0.553	0.618	0.707
30 min	0.311	0.407	0.486	0.593	0.675	0.760	0.849	0.971
60 min	0.430	0.563	0.671	0.819	0.934	1.051	1.174	1.343
3 hour	0.674	0.837	0.977	1.180	1.346	1.524	1.718	2.000
6 hour	0.893	1.097	1.274	1.529	1.740	1.967	2.215	2.576
12 hour	1.175	1.463	1.703	2.041	2.309	2.589	2.886	3.302
24 hour	1.621	2.050	2.397	2.868	3.229	3.595	3.972	4.482
48 hour	2.014	2.529	2.945	3.503	3.929	4.358	4.797	5.388
72 hour	2.273	2.839	3.295	3.908	4.375	4.847	5.329	5.979
96 hour	2.490	3.101	3.593	4.253	4.754	5.258	5.773	6.465

Upper Watershed Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.615	0.616	0.613	0.614	0.611	0.613	0.613	0.612
10 min	0.618	0.618	0.614	0.615	0.613	0.612	0.612	0.610
15 min	0.618	0.616	0.614	0.614	0.612	0.612	0.612	0.612
30 min	0.615	0.615	0.614	0.612	0.612	0.611	0.611	0.610
60 min	0.618	0.616	0.616	0.615	0.614	0.614	0.613	0.612
3 hour	0.614	0.612	0.610	0.609	0.609	0.608	0.609	0.609
6 hour	0.611	0.608	0.607	0.606	0.605	0.606	0.606	0.607
12 hour	0.609	0.606	0.604	0.603	0.603	0.602	0.602	0.602
24 hour	0.609	0.607	0.606	0.605	0.604	0.604	0.603	0.603
48 hour	0.607	0.606	0.604	0.604	0.604	0.604	0.604	0.604
72 hour	0.605	0.603	0.602	0.602	0.602	0.603	0.603	0.604
96 hour	0.605	0.603	0.602	0.602	0.602	0.603	0.603	0.605

Upper Watershed Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.081	0.105	0.125	0.153	0.174	0.196	0.218	0.250
10 min	0.116	0.151	0.179	0.220	0.249	0.280	0.313	0.356
15 min	0.140	0.182	0.217	0.265	0.300	0.338	0.378	0.433
30 min	0.191	0.250	0.298	0.363	0.413	0.464	0.519	0.592
60 min	0.266	0.347	0.413	0.504	0.573	0.645	0.720	0.822
3 hour	0.414	0.512	0.596	0.719	0.820	0.927	1.046	1.218
6 hour	0.546	0.667	0.773	0.927	1.053	1.192	1.342	1.564
12 hour	0.716	0.887	1.029	1.231	1.392	1.559	1.737	1.988
24 hour	0.987	1.244	1.453	1.735	1.950	2.171	2.395	2.703
48 hour	1.222	1.533	1.779	2.116	2.373	2.632	2.897	3.254
72 hour	1.375	1.712	1.984	2.353	2.634	2.923	3.213	3.611
96 hour	1.506	1.870	2.163	2.560	2.862	3.171	3.481	3.911

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: SCK

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.171	0.204	0.249	0.284	0.319	0.356	0.408
10 min	0.187	0.245	0.292	0.357	0.406	0.458	0.511	0.584
15 min	0.227	0.296	0.353	0.431	0.491	0.553	0.618	0.707
30 min	0.311	0.407	0.486	0.593	0.675	0.760	0.849	0.971
60 min	0.430	0.563	0.671	0.819	0.934	1.051	1.174	1.343
3 hour	0.674	0.837	0.977	1.180	1.346	1.524	1.718	2.000
6 hour	0.893	1.097	1.274	1.529	1.740	1.967	2.215	2.576
12 hour	1.175	1.463	1.703	2.041	2.309	2.589	2.886	3.302
24 hour	1.621	2.050	2.397	2.868	3.229	3.595	3.972	4.482
48 hour	2.014	2.529	2.945	3.503	3.929	4.358	4.797	5.388
72 hour	2.273	2.839	3.295	3.908	4.375	4.847	5.329	5.979
96 hour	2.490	3.101	3.593	4.253	4.754	5.258	5.773	6.465

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.610	0.611	0.608	0.609	0.606	0.608	0.608	0.607
10 min	0.614	0.613	0.609	0.610	0.608	0.607	0.607	0.605
15 min	0.613	0.612	0.609	0.609	0.607	0.607	0.607	0.607
30 min	0.610	0.610	0.609	0.607	0.607	0.606	0.606	0.605
60 min	0.614	0.611	0.611	0.610	0.609	0.609	0.608	0.607
3 hour	0.609	0.607	0.605	0.604	0.604	0.603	0.604	0.604
6 hour	0.606	0.603	0.601	0.601	0.600	0.600	0.600	0.601
12 hour	0.605	0.601	0.599	0.597	0.597	0.596	0.596	0.596
24 hour	0.604	0.602	0.601	0.599	0.598	0.598	0.597	0.597
48 hour	0.603	0.601	0.599	0.599	0.599	0.599	0.599	0.599
72 hour	0.600	0.598	0.597	0.597	0.597	0.597	0.598	0.599
96 hour	0.600	0.598	0.597	0.597	0.597	0.598	0.598	0.600

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.080	0.104	0.124	0.152	0.172	0.194	0.216	0.248
10 min	0.115	0.150	0.178	0.218	0.247	0.278	0.310	0.353
15 min	0.139	0.181	0.215	0.262	0.298	0.336	0.375	0.429
30 min	0.190	0.248	0.296	0.360	0.410	0.461	0.514	0.587
60 min	0.264	0.344	0.410	0.500	0.569	0.640	0.714	0.815
3 hour	0.410	0.508	0.591	0.713	0.813	0.919	1.038	1.208
6 hour	0.541	0.661	0.766	0.919	1.044	1.180	1.329	1.548
12 hour	0.711	0.879	1.020	1.218	1.378	1.543	1.720	1.968
24 hour	0.979	1.234	1.441	1.718	1.931	2.150	2.371	2.676
48 hour	1.214	1.520	1.764	2.098	2.353	2.610	2.873	3.227
72 hour	1.364	1.698	1.967	2.333	2.612	2.894	3.187	3.581
96 hour	1.494	1.854	2.145	2.539	2.838	3.144	3.452	3.879

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FLW

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.137	0.182	0.220	0.273	0.316	0.360	0.407	0.474
10 min	0.196	0.261	0.316	0.392	0.453	0.516	0.584	0.680
15 min	0.237	0.316	0.382	0.474	0.547	0.624	0.706	0.822
30 min	0.328	0.437	0.528	0.656	0.758	0.864	0.978	1.138
60 min	0.444	0.592	0.715	0.888	1.026	1.170	1.324	1.541
3 hour	0.740	0.949	1.125	1.373	1.572	1.782	2.005	2.320
6 hour	1.021	1.296	1.528	1.854	2.115	2.389	2.681	3.092
12 hour	1.393	1.781	2.106	2.561	2.921	3.298	3.696	4.254
24 hour	1.936	2.494	2.955	3.590	4.085	4.597	5.132	5.871
48 hour	2.491	3.180	3.736	4.483	5.053	5.629	6.218	7.013
72 hour	2.910	3.694	4.318	5.147	5.769	6.390	7.018	7.852
96 hour	3.220	4.072	4.746	5.631	6.291	6.943	7.598	8.459

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.658	0.657	0.657	0.656	0.659	0.660	0.661	0.662
10 min	0.659	0.655	0.658	0.657	0.657	0.660	0.661	0.662
15 min	0.658	0.657	0.656	0.657	0.658	0.659	0.661	0.662
30 min	0.657	0.657	0.656	0.658	0.658	0.659	0.661	0.663
60 min	0.656	0.655	0.656	0.657	0.658	0.659	0.660	0.662
3 hour	0.661	0.660	0.659	0.660	0.660	0.660	0.660	0.660
6 hour	0.663	0.663	0.663	0.662	0.662	0.662	0.661	0.661
12 hour	0.665	0.664	0.664	0.663	0.663	0.663	0.663	0.664
24 hour	0.666	0.664	0.664	0.664	0.664	0.664	0.665	0.666
48 hour	0.669	0.668	0.667	0.667	0.667	0.667	0.667	0.667
72 hour	0.671	0.670	0.669	0.669	0.669	0.669	0.669	0.669
96 hour	0.672	0.671	0.671	0.671	0.670	0.670	0.670	0.670

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.090	0.120	0.145	0.179	0.208	0.238	0.269	0.314
10 min	0.129	0.171	0.208	0.258	0.298	0.341	0.386	0.450
15 min	0.156	0.208	0.251	0.311	0.360	0.411	0.467	0.544
30 min	0.215	0.287	0.346	0.432	0.499	0.569	0.646	0.754
60 min	0.291	0.388	0.469	0.583	0.675	0.771	0.874	1.020
3 hour	0.489	0.626	0.741	0.906	1.038	1.176	1.323	1.531
6 hour	0.677	0.859	1.013	1.227	1.400	1.582	1.772	2.044
12 hour	0.926	1.183	1.398	1.698	1.937	2.187	2.450	2.825
24 hour	1.289	1.656	1.962	2.384	2.712	3.052	3.413	3.910
48 hour	1.666	2.124	2.492	2.990	3.370	3.755	4.147	4.678
72 hour	1.953	2.475	2.889	3.443	3.859	4.275	4.695	5.253
96 hour	2.164	2.732	3.185	3.778	4.215	4.652	5.091	5.668

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FLW

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.137	0.182	0.220	0.273	0.316	0.360	0.407	0.474
10 min	0.196	0.261	0.316	0.392	0.453	0.516	0.584	0.680
15 min	0.237	0.316	0.382	0.474	0.547	0.624	0.706	0.822
30 min	0.328	0.437	0.528	0.656	0.758	0.864	0.978	1.138
60 min	0.444	0.592	0.715	0.888	1.026	1.170	1.324	1.541
3 hour	0.740	0.949	1.125	1.373	1.572	1.782	2.005	2.320
6 hour	1.021	1.296	1.528	1.854	2.115	2.389	2.681	3.092
12 hour	1.393	1.781	2.106	2.561	2.921	3.298	3.696	4.254
24 hour	1.936	2.494	2.955	3.590	4.085	4.597	5.132	5.871
48 hour	2.491	3.180	3.736	4.483	5.053	5.629	6.218	7.013
72 hour	2.910	3.694	4.318	5.147	5.769	6.390	7.018	7.852
96 hour	3.220	4.072	4.746	5.631	6.291	6.943	7.598	8.459

Farmington Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.721	0.720	0.719	0.719	0.720	0.721	0.722	0.723
10 min	0.722	0.719	0.720	0.719	0.719	0.721	0.721	0.723
15 min	0.721	0.720	0.718	0.719	0.720	0.720	0.721	0.723
30 min	0.720	0.719	0.719	0.719	0.719	0.720	0.721	0.723
60 min	0.720	0.718	0.719	0.719	0.720	0.720	0.721	0.723
3 hour	0.723	0.721	0.720	0.720	0.720	0.720	0.720	0.720
6 hour	0.724	0.723	0.722	0.721	0.721	0.720	0.720	0.720
12 hour	0.725	0.723	0.722	0.721	0.721	0.721	0.721	0.721
24 hour	0.726	0.724	0.722	0.722	0.722	0.722	0.722	0.722
48 hour	0.728	0.726	0.725	0.725	0.724	0.724	0.724	0.725
72 hour	0.729	0.728	0.727	0.726	0.726	0.726	0.726	0.726
96 hour	0.730	0.729	0.728	0.728	0.728	0.727	0.727	0.727

Farmington Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.099	0.131	0.158	0.196	0.228	0.260	0.294	0.343
10 min	0.142	0.188	0.228	0.282	0.326	0.372	0.421	0.492
15 min	0.171	0.228	0.274	0.341	0.394	0.449	0.509	0.594
30 min	0.236	0.314	0.380	0.472	0.545	0.622	0.705	0.823
60 min	0.320	0.425	0.514	0.638	0.739	0.842	0.955	1.114
3 hour	0.535	0.684	0.810	0.989	1.132	1.283	1.444	1.670
6 hour	0.739	0.937	1.103	1.337	1.525	1.720	1.930	2.226
12 hour	1.010	1.288	1.521	1.846	2.106	2.378	2.665	3.067
24 hour	1.406	1.806	2.134	2.592	2.949	3.319	3.705	4.239
48 hour	1.813	2.309	2.709	3.250	3.658	4.075	4.502	5.084
72 hour	2.121	2.689	3.139	3.737	4.188	4.639	5.095	5.701
96 hour	2.351	2.968	3.455	4.099	4.580	5.048	5.524	6.150

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FLW

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.137	0.182	0.220	0.273	0.316	0.360	0.407	0.474
10 min	0.196	0.261	0.316	0.392	0.453	0.516	0.584	0.680
15 min	0.237	0.316	0.382	0.474	0.547	0.624	0.706	0.822
30 min	0.328	0.437	0.528	0.656	0.758	0.864	0.978	1.138
60 min	0.444	0.592	0.715	0.888	1.026	1.170	1.324	1.541
3 hour	0.740	0.949	1.125	1.373	1.572	1.782	2.005	2.320
6 hour	1.021	1.296	1.528	1.854	2.115	2.389	2.681	3.092
12 hour	1.393	1.781	2.106	2.561	2.921	3.298	3.696	4.254
24 hour	1.936	2.494	2.955	3.590	4.085	4.597	5.132	5.871
48 hour	2.491	3.180	3.736	4.483	5.053	5.629	6.218	7.013
72 hour	2.910	3.694	4.318	5.147	5.769	6.390	7.018	7.852
96 hour	3.220	4.072	4.746	5.631	6.291	6.943	7.598	8.459

Upper Watershed Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.779	0.779	0.780	0.780	0.781	0.780	0.781	0.780
10 min	0.780	0.780	0.781	0.780	0.780	0.781	0.780	0.780
15 min	0.779	0.780	0.780	0.780	0.780	0.780	0.780	0.780
30 min	0.778	0.779	0.780	0.780	0.780	0.780	0.780	0.780
60 min	0.779	0.780	0.781	0.781	0.781	0.781	0.781	0.780
3 hour	0.779	0.780	0.780	0.780	0.781	0.780	0.780	0.780
6 hour	0.778	0.779	0.780	0.780	0.780	0.780	0.780	0.780
12 hour	0.778	0.779	0.780	0.781	0.781	0.782	0.782	0.782
24 hour	0.777	0.779	0.780	0.781	0.781	0.782	0.783	0.783
48 hour	0.777	0.778	0.779	0.780	0.780	0.781	0.782	0.782
72 hour	0.777	0.778	0.778	0.779	0.780	0.780	0.781	0.781
96 hour	0.776	0.777	0.778	0.778	0.779	0.779	0.780	0.780

Upper Watershed Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.107	0.142	0.172	0.213	0.247	0.281	0.318	0.370
10 min	0.153	0.204	0.247	0.306	0.353	0.403	0.456	0.530
15 min	0.185	0.246	0.298	0.370	0.427	0.487	0.551	0.641
30 min	0.255	0.340	0.412	0.512	0.591	0.674	0.763	0.888
60 min	0.346	0.462	0.558	0.694	0.801	0.914	1.034	1.202
3 hour	0.576	0.740	0.878	1.071	1.228	1.390	1.564	1.810
6 hour	0.794	1.010	1.192	1.446	1.650	1.863	2.091	2.412
12 hour	1.084	1.387	1.643	2.000	2.281	2.579	2.890	3.327
24 hour	1.504	1.943	2.305	2.804	3.190	3.595	4.018	4.597
48 hour	1.936	2.474	2.910	3.497	3.941	4.396	4.862	5.484
72 hour	2.261	2.874	3.359	4.010	4.500	4.984	5.481	6.132
96 hour	2.499	3.164	3.692	4.381	4.901	5.409	5.926	6.598

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FLW

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.137	0.182	0.220	0.273	0.316	0.360	0.407	0.474
10 min	0.196	0.261	0.316	0.392	0.453	0.516	0.584	0.680
15 min	0.237	0.316	0.382	0.474	0.547	0.624	0.706	0.822
30 min	0.328	0.437	0.528	0.656	0.758	0.864	0.978	1.138
60 min	0.444	0.592	0.715	0.888	1.026	1.170	1.324	1.541
3 hour	0.740	0.949	1.125	1.373	1.572	1.782	2.005	2.320
6 hour	1.021	1.296	1.528	1.854	2.115	2.389	2.681	3.092
12 hour	1.393	1.781	2.106	2.561	2.921	3.298	3.696	4.254
24 hour	1.936	2.494	2.955	3.590	4.085	4.597	5.132	5.871
48 hour	2.491	3.180	3.736	4.483	5.053	5.629	6.218	7.013
72 hour	2.910	3.694	4.318	5.147	5.769	6.390	7.018	7.852
96 hour	3.220	4.072	4.746	5.631	6.291	6.943	7.598	8.459

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.750	0.750	0.750	0.750	0.751	0.751	0.752	0.752
10 min	0.751	0.750	0.751	0.750	0.750	0.751	0.751	0.752
15 min	0.750	0.750	0.749	0.750	0.750	0.750	0.751	0.752
30 min	0.749	0.749	0.750	0.750	0.750	0.750	0.751	0.752
60 min	0.750	0.749	0.750	0.750	0.751	0.751	0.751	0.752
3 hour	0.751	0.751	0.750	0.750	0.751	0.750	0.750	0.750
6 hour	0.751	0.751	0.751	0.751	0.751	0.750	0.750	0.750
12 hour	0.752	0.751	0.751	0.751	0.751	0.752	0.752	0.752
24 hour	0.752	0.752	0.751	0.752	0.752	0.752	0.753	0.753
48 hour	0.753	0.752	0.752	0.753	0.752	0.753	0.753	0.754
72 hour	0.753	0.753	0.753	0.753	0.753	0.753	0.754	0.754
96 hour	0.753	0.753	0.753	0.753	0.754	0.753	0.754	0.754

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.103	0.137	0.165	0.205	0.237	0.270	0.306	0.356
10 min	0.147	0.196	0.237	0.294	0.340	0.388	0.439	0.511
15 min	0.178	0.237	0.286	0.356	0.410	0.468	0.530	0.618
30 min	0.246	0.327	0.396	0.492	0.569	0.648	0.734	0.856
60 min	0.333	0.443	0.536	0.666	0.771	0.879	0.994	1.159
3 hour	0.556	0.713	0.844	1.030	1.181	1.337	1.504	1.740
6 hour	0.767	0.973	1.148	1.392	1.588	1.792	2.011	2.319
12 hour	1.048	1.338	1.582	1.923	2.194	2.480	2.779	3.199
24 hour	1.456	1.875	2.219	2.700	3.072	3.457	3.864	4.421
48 hour	1.876	2.391	2.809	3.376	3.800	4.239	4.682	5.288
72 hour	2.191	2.782	3.251	3.876	4.344	4.812	5.292	5.920
96 hour	2.425	3.066	3.574	4.240	4.743	5.228	5.729	6.378

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: NHG

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.168	0.218	0.261	0.323	0.372	0.425	0.482	0.565
10 min	0.241	0.313	0.375	0.462	0.534	0.609	0.691	0.810
15 min	0.292	0.379	0.453	0.559	0.645	0.737	0.836	0.979
30 min	0.406	0.527	0.631	0.778	0.898	1.026	1.164	1.364
60 min	0.542	0.703	0.841	1.038	1.198	1.368	1.552	1.817
3 hour	0.900	1.126	1.322	1.608	1.844	2.100	2.381	2.793
6 hour	1.267	1.567	1.824	2.194	2.495	2.818	3.168	3.673
12 hour	1.751	2.165	2.511	2.993	3.375	3.772	4.193	4.784
24 hour	2.467	3.062	3.544	4.196	4.694	5.200	5.721	6.428
48 hour	3.189	3.954	4.558	5.353	5.944	6.530	7.120	7.901
72 hour	3.740	4.636	5.334	6.238	6.902	7.552	8.198	9.042
96 hour	4.188	5.191	5.964	6.953	7.671	8.366	9.052	9.937

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.637	0.637	0.635	0.635	0.635	0.638	0.637	0.638
10 min	0.637	0.635	0.634	0.635	0.635	0.637	0.638	0.639
15 min	0.636	0.636	0.635	0.636	0.636	0.637	0.638	0.639
30 min	0.638	0.636	0.635	0.636	0.636	0.637	0.638	0.639
60 min	0.637	0.635	0.634	0.635	0.635	0.636	0.637	0.638
3 hour	0.638	0.637	0.636	0.636	0.636	0.637	0.637	0.638
6 hour	0.641	0.639	0.639	0.638	0.638	0.638	0.638	0.638
12 hour	0.643	0.640	0.639	0.638	0.638	0.637	0.637	0.637
24 hour	0.644	0.642	0.640	0.639	0.638	0.637	0.637	0.636
48 hour	0.646	0.644	0.643	0.642	0.641	0.640	0.640	0.639
72 hour	0.647	0.646	0.645	0.643	0.643	0.642	0.641	0.641
96 hour	0.648	0.647	0.646	0.645	0.644	0.644	0.643	0.642

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.107	0.139	0.166	0.205	0.236	0.271	0.307	0.360
10 min	0.154	0.199	0.238	0.293	0.339	0.388	0.441	0.518
15 min	0.186	0.241	0.288	0.356	0.410	0.469	0.533	0.626
30 min	0.259	0.335	0.401	0.495	0.571	0.654	0.743	0.872
60 min	0.345	0.446	0.533	0.659	0.761	0.870	0.989	1.159
3 hour	0.574	0.717	0.841	1.023	1.173	1.338	1.517	1.782
6 hour	0.812	1.001	1.166	1.400	1.592	1.798	2.021	2.343
12 hour	1.126	1.386	1.605	1.910	2.153	2.403	2.671	3.047
24 hour	1.589	1.966	2.268	2.681	2.995	3.312	3.644	4.088
48 hour	2.060	2.546	2.931	3.437	3.810	4.179	4.557	5.049
72 hour	2.420	2.995	3.440	4.011	4.438	4.848	5.255	5.796
96 hour	2.714	3.359	3.853	4.485	4.940	5.388	5.820	6.380

FRENCH CAMP SLOUGH

NOAA14 Precipitation Frequency Depths

Rainfall Zone: NHG

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.168	0.218	0.261	0.323	0.372	0.425	0.482	0.565
10 min	0.241	0.313	0.375	0.462	0.534	0.609	0.691	0.810
15 min	0.292	0.379	0.453	0.559	0.645	0.737	0.836	0.979
30 min	0.406	0.527	0.631	0.778	0.898	1.026	1.164	1.364
60 min	0.542	0.703	0.841	1.038	1.198	1.368	1.552	1.817
3 hour	0.900	1.126	1.322	1.608	1.844	2.100	2.381	2.793
6 hour	1.267	1.567	1.824	2.194	2.495	2.818	3.168	3.673
12 hour	1.751	2.165	2.511	2.993	3.375	3.772	4.193	4.784
24 hour	2.467	3.062	3.544	4.196	4.694	5.200	5.721	6.428
48 hour	3.189	3.954	4.558	5.353	5.944	6.530	7.120	7.901
72 hour	3.740	4.636	5.334	6.238	6.902	7.552	8.198	9.042
96 hour	4.188	5.191	5.964	6.953	7.671	8.366	9.052	9.937

Farmington Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.637	0.637	0.635	0.635	0.635	0.638	0.637	0.638
10 min	0.637	0.635	0.634	0.635	0.635	0.637	0.638	0.639
15 min	0.636	0.636	0.635	0.636	0.636	0.637	0.638	0.639
30 min	0.638	0.636	0.635	0.636	0.636	0.637	0.638	0.639
60 min	0.637	0.635	0.634	0.635	0.635	0.636	0.637	0.638
3 hour	0.638	0.637	0.636	0.636	0.636	0.637	0.637	0.638
6 hour	0.641	0.639	0.639	0.638	0.638	0.638	0.638	0.638
12 hour	0.643	0.640	0.639	0.638	0.638	0.637	0.637	0.637
24 hour	0.644	0.642	0.640	0.639	0.638	0.637	0.637	0.636
48 hour	0.646	0.644	0.643	0.642	0.641	0.640	0.640	0.639
72 hour	0.647	0.646	0.645	0.643	0.643	0.642	0.641	0.641
96 hour	0.648	0.647	0.646	0.645	0.644	0.644	0.643	0.642

Farmington Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.107	0.139	0.166	0.205	0.236	0.271	0.307	0.360
10 min	0.154	0.199	0.238	0.293	0.339	0.388	0.441	0.518
15 min	0.186	0.241	0.288	0.356	0.410	0.469	0.533	0.626
30 min	0.259	0.335	0.401	0.495	0.571	0.654	0.743	0.872
60 min	0.345	0.446	0.533	0.659	0.761	0.870	0.989	1.159
3 hour	0.574	0.717	0.841	1.023	1.173	1.338	1.517	1.782
6 hour	0.812	1.001	1.166	1.400	1.592	1.798	2.021	2.343
12 hour	1.126	1.386	1.605	1.910	2.153	2.403	2.671	3.047
24 hour	1.589	1.966	2.268	2.681	2.995	3.312	3.644	4.088
48 hour	2.060	2.546	2.931	3.437	3.810	4.179	4.557	5.049
72 hour	2.420	2.995	3.440	4.011	4.438	4.848	5.255	5.796
96 hour	2.714	3.359	3.853	4.485	4.940	5.388	5.820	6.380

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: NHG

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.168	0.218	0.261	0.323	0.372	0.425	0.482	0.565
10 min	0.241	0.313	0.375	0.462	0.534	0.609	0.691	0.810
15 min	0.292	0.379	0.453	0.559	0.645	0.737	0.836	0.979
30 min	0.406	0.527	0.631	0.778	0.898	1.026	1.164	1.364
60 min	0.542	0.703	0.841	1.038	1.198	1.368	1.552	1.817
3 hour	0.900	1.126	1.322	1.608	1.844	2.100	2.381	2.793
6 hour	1.267	1.567	1.824	2.194	2.495	2.818	3.168	3.673
12 hour	1.751	2.165	2.511	2.993	3.375	3.772	4.193	4.784
24 hour	2.467	3.062	3.544	4.196	4.694	5.200	5.721	6.428
48 hour	3.189	3.954	4.558	5.353	5.944	6.530	7.120	7.901
72 hour	3.740	4.636	5.334	6.238	6.902	7.552	8.198	9.042
96 hour	4.188	5.191	5.964	6.953	7.671	8.366	9.052	9.937

Upper Watershed Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
10 min	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
15 min	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
30 min	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
60 min	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
3 hour	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
6 hour	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
12 hour	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
24 hour	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
48 hour	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
72 hour	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924
96 hour	0.924	0.924	0.924	0.924	0.924	0.924	0.924	0.924

Upper Watershed Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.155	0.201	0.241	0.298	0.344	0.393	0.445	0.522
10 min	0.223	0.289	0.347	0.427	0.493	0.563	0.638	0.748
15 min	0.270	0.350	0.419	0.517	0.596	0.681	0.772	0.905
30 min	0.375	0.487	0.583	0.719	0.830	0.948	1.076	1.260
60 min	0.501	0.650	0.777	0.959	1.107	1.264	1.434	1.679
3 hour	0.832	1.040	1.222	1.486	1.704	1.940	2.200	2.581
6 hour	1.171	1.448	1.685	2.027	2.305	2.604	2.927	3.394
12 hour	1.618	2.000	2.320	2.766	3.119	3.485	3.874	4.420
24 hour	2.280	2.829	3.275	3.877	4.337	4.805	5.286	5.939
48 hour	2.947	3.653	4.212	4.946	5.492	6.034	6.579	7.301
72 hour	3.456	4.284	4.929	5.764	6.377	6.978	7.575	8.355
96 hour	3.870	4.796	5.511	6.425	7.088	7.730	8.364	9.182

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: NHG

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.168	0.218	0.261	0.323	0.372	0.425	0.482	0.565
10 min	0.241	0.313	0.375	0.462	0.534	0.609	0.691	0.810
15 min	0.292	0.379	0.453	0.559	0.645	0.737	0.836	0.979
30 min	0.406	0.527	0.631	0.778	0.898	1.026	1.164	1.364
60 min	0.542	0.703	0.841	1.038	1.198	1.368	1.552	1.817
3 hour	0.900	1.126	1.322	1.608	1.844	2.100	2.381	2.793
6 hour	1.267	1.567	1.824	2.194	2.495	2.818	3.168	3.673
12 hour	1.751	2.165	2.511	2.993	3.375	3.772	4.193	4.784
24 hour	2.467	3.062	3.544	4.196	4.694	5.200	5.721	6.428
48 hour	3.189	3.954	4.558	5.353	5.944	6.530	7.120	7.901
72 hour	3.740	4.636	5.334	6.238	6.902	7.552	8.198	9.042
96 hour	4.188	5.191	5.964	6.953	7.671	8.366	9.052	9.937

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.781	0.781	0.780	0.780	0.780	0.781	0.781	0.781
10 min	0.781	0.780	0.779	0.780	0.780	0.781	0.781	0.782
15 min	0.780	0.780	0.780	0.780	0.780	0.781	0.781	0.782
30 min	0.781	0.780	0.780	0.780	0.780	0.781	0.781	0.782
60 min	0.781	0.780	0.779	0.780	0.780	0.780	0.781	0.781
3 hour	0.781	0.781	0.780	0.780	0.780	0.781	0.781	0.781
6 hour	0.783	0.782	0.782	0.781	0.781	0.781	0.781	0.781
12 hour	0.784	0.782	0.782	0.781	0.781	0.781	0.781	0.781
24 hour	0.784	0.783	0.782	0.782	0.781	0.781	0.781	0.780
48 hour	0.785	0.784	0.784	0.783	0.783	0.782	0.782	0.782
72 hour	0.786	0.785	0.785	0.784	0.784	0.783	0.783	0.783
96 hour	0.786	0.786	0.785	0.785	0.784	0.784	0.784	0.783

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.131	0.170	0.204	0.252	0.290	0.332	0.376	0.441
10 min	0.188	0.244	0.292	0.360	0.417	0.476	0.540	0.633
15 min	0.228	0.296	0.353	0.436	0.503	0.576	0.653	0.766
30 min	0.317	0.411	0.492	0.607	0.700	0.801	0.909	1.067
60 min	0.423	0.548	0.655	0.810	0.934	1.067	1.212	1.419
3 hour	0.703	0.879	1.031	1.254	1.438	1.640	1.860	2.181
6 hour	0.992	1.225	1.426	1.714	1.949	2.201	2.474	2.869
12 hour	1.373	1.693	1.964	2.338	2.636	2.946	3.275	3.736
24 hour	1.934	2.398	2.771	3.281	3.666	4.061	4.468	5.014
48 hour	2.503	3.100	3.573	4.191	4.654	5.106	5.568	6.179
72 hour	2.940	3.639	4.187	4.891	5.411	5.913	6.419	7.080
96 hour	3.292	4.080	4.682	5.458	6.014	6.559	7.097	7.781

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FRM

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.127	0.174	0.211	0.260	0.298	0.335	0.373	0.424
10 min	0.182	0.249	0.302	0.373	0.427	0.480	0.534	0.607
15 min	0.220	0.301	0.365	0.451	0.516	0.581	0.646	0.734
30 min	0.306	0.418	0.508	0.627	0.717	0.808	0.899	1.021
60 min	0.415	0.567	0.689	0.850	0.972	1.094	1.218	1.384
3 hour	0.664	0.874	1.049	1.292	1.484	1.683	1.892	2.184
6 hour	0.889	1.163	1.393	1.714	1.968	2.234	2.515	2.908
12 hour	1.188	1.585	1.910	2.357	2.704	3.059	3.428	3.931
24 hour	1.619	2.190	2.648	3.260	3.722	4.184	4.654	5.280
48 hour	2.005	2.665	3.186	3.870	4.378	4.880	5.381	6.038
72 hour	2.286	3.004	3.566	4.298	4.839	5.369	5.896	6.581
96 hour	2.468	3.217	3.802	4.561	5.118	5.663	6.203	6.902

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.731	0.731	0.733	0.733	0.734	0.733	0.732	0.732
10 min	0.728	0.731	0.733	0.732	0.733	0.733	0.733	0.734
15 min	0.729	0.731	0.734	0.733	0.734	0.734	0.733	0.732
30 min	0.731	0.732	0.733	0.734	0.734	0.734	0.734	0.734
60 min	0.729	0.732	0.733	0.733	0.733	0.733	0.733	0.733
3 hour	0.730	0.733	0.734	0.735	0.735	0.736	0.736	0.735
6 hour	0.731	0.734	0.735	0.736	0.737	0.737	0.737	0.737
12 hour	0.732	0.735	0.737	0.738	0.738	0.739	0.739	0.739
24 hour	0.731	0.734	0.736	0.737	0.738	0.738	0.738	0.739
48 hour	0.731	0.734	0.735	0.736	0.736	0.736	0.736	0.736
72 hour	0.732	0.734	0.735	0.736	0.736	0.736	0.736	0.736
96 hour	0.731	0.733	0.734	0.734	0.735	0.735	0.735	0.734

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.093	0.127	0.155	0.191	0.219	0.246	0.273	0.310
10 min	0.132	0.182	0.221	0.273	0.313	0.352	0.391	0.446
15 min	0.160	0.220	0.268	0.331	0.379	0.426	0.474	0.537
30 min	0.224	0.306	0.372	0.460	0.526	0.593	0.660	0.749
60 min	0.303	0.415	0.505	0.623	0.712	0.802	0.893	1.014
3 hour	0.485	0.641	0.770	0.950	1.091	1.239	1.393	1.605
6 hour	0.650	0.854	1.024	1.262	1.450	1.646	1.854	2.143
12 hour	0.870	1.165	1.408	1.739	1.996	2.261	2.533	2.905
24 hour	1.183	1.607	1.949	2.403	2.747	3.088	3.435	3.902
48 hour	1.466	1.956	2.342	2.848	3.222	3.592	3.960	4.444
72 hour	1.673	2.205	2.621	3.163	3.562	3.952	4.339	4.844
96 hour	1.804	2.358	2.791	3.348	3.762	4.162	4.559	5.066

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FRM

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.127	0.174	0.211	0.260	0.298	0.335	0.373	0.424
10 min	0.182	0.249	0.302	0.373	0.427	0.480	0.534	0.607
15 min	0.220	0.301	0.365	0.451	0.516	0.581	0.646	0.734
30 min	0.306	0.418	0.508	0.627	0.717	0.808	0.899	1.021
60 min	0.415	0.567	0.689	0.850	0.972	1.094	1.218	1.384
3 hour	0.664	0.874	1.049	1.292	1.484	1.683	1.892	2.184
6 hour	0.889	1.163	1.393	1.714	1.968	2.234	2.515	2.908
12 hour	1.188	1.585	1.910	2.357	2.704	3.059	3.428	3.931
24 hour	1.619	2.190	2.648	3.260	3.722	4.184	4.654	5.280
48 hour	2.005	2.665	3.186	3.870	4.378	4.880	5.381	6.038
72 hour	2.286	3.004	3.566	4.298	4.839	5.369	5.896	6.581
96 hour	2.468	3.217	3.802	4.561	5.118	5.663	6.203	6.902

Farmington Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
10 min	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
15 min	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
30 min	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
60 min	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
3 hour	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
6 hour	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
12 hour	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
24 hour	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
48 hour	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
72 hour	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822
96 hour	0.822	0.822	0.822	0.822	0.822	0.822	0.822	0.822

Farmington Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.104	0.143	0.173	0.214	0.245	0.275	0.307	0.349
10 min	0.150	0.205	0.248	0.307	0.351	0.395	0.439	0.499
15 min	0.181	0.247	0.300	0.371	0.424	0.478	0.531	0.603
30 min	0.252	0.344	0.418	0.515	0.589	0.664	0.739	0.839
60 min	0.341	0.466	0.566	0.699	0.799	0.899	1.001	1.138
3 hour	0.546	0.718	0.862	1.062	1.220	1.383	1.555	1.795
6 hour	0.731	0.956	1.145	1.409	1.618	1.836	2.067	2.390
12 hour	0.977	1.303	1.570	1.937	2.223	2.514	2.818	3.231
24 hour	1.331	1.800	2.177	2.680	3.059	3.439	3.826	4.340
48 hour	1.648	2.191	2.619	3.181	3.599	4.011	4.423	4.963
72 hour	1.879	2.469	2.931	3.533	3.978	4.413	4.847	5.410
96 hour	2.029	2.644	3.125	3.749	4.207	4.655	5.099	5.673

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FRM

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.127	0.174	0.211	0.260	0.298	0.335	0.373	0.424
10 min	0.182	0.249	0.302	0.373	0.427	0.480	0.534	0.607
15 min	0.220	0.301	0.365	0.451	0.516	0.581	0.646	0.734
30 min	0.306	0.418	0.508	0.627	0.717	0.808	0.899	1.021
60 min	0.415	0.567	0.689	0.850	0.972	1.094	1.218	1.384
3 hour	0.664	0.874	1.049	1.292	1.484	1.683	1.892	2.184
6 hour	0.889	1.163	1.393	1.714	1.968	2.234	2.515	2.908
12 hour	1.188	1.585	1.910	2.357	2.704	3.059	3.428	3.931
24 hour	1.619	2.190	2.648	3.260	3.722	4.184	4.654	5.280
48 hour	2.005	2.665	3.186	3.870	4.378	4.880	5.381	6.038
72 hour	2.286	3.004	3.566	4.298	4.839	5.369	5.896	6.581
96 hour	2.468	3.217	3.802	4.561	5.118	5.663	6.203	6.902

Upper Watershed Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.620	0.622	0.624	0.625	0.625	0.622	0.621	0.619
10 min	0.618	0.625	0.623	0.624	0.625	0.623	0.621	0.620
15 min	0.618	0.623	0.627	0.625	0.625	0.624	0.622	0.619
30 min	0.620	0.624	0.625	0.625	0.625	0.624	0.622	0.620
60 min	0.619	0.625	0.626	0.626	0.625	0.624	0.623	0.620
3 hour	0.616	0.620	0.623	0.624	0.625	0.625	0.625	0.624
6 hour	0.613	0.618	0.621	0.623	0.624	0.624	0.625	0.625
12 hour	0.612	0.618	0.621	0.624	0.625	0.626	0.626	0.627
24 hour	0.609	0.617	0.621	0.623	0.624	0.625	0.626	0.626
48 hour	0.606	0.612	0.616	0.618	0.619	0.620	0.620	0.621
72 hour	0.604	0.609	0.612	0.614	0.615	0.616	0.617	0.617
96 hour	0.601	0.605	0.608	0.610	0.611	0.612	0.612	0.613

Upper Watershed Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.079	0.108	0.132	0.163	0.186	0.208	0.232	0.262
10 min	0.112	0.156	0.188	0.233	0.267	0.299	0.332	0.376
15 min	0.136	0.188	0.229	0.282	0.323	0.363	0.402	0.454
30 min	0.190	0.261	0.318	0.392	0.448	0.504	0.559	0.633
60 min	0.257	0.354	0.431	0.532	0.608	0.683	0.759	0.858
3 hour	0.409	0.542	0.654	0.806	0.928	1.052	1.183	1.363
6 hour	0.545	0.719	0.865	1.068	1.228	1.394	1.572	1.818
12 hour	0.727	0.980	1.186	1.471	1.690	1.915	2.146	2.465
24 hour	0.986	1.351	1.644	2.031	2.323	2.615	2.913	3.305
48 hour	1.215	1.631	1.963	2.392	2.710	3.026	3.336	3.750
72 hour	1.381	1.829	2.182	2.639	2.976	3.307	3.638	4.060
96 hour	1.483	1.946	2.312	2.782	3.127	3.466	3.796	4.231

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: FRM

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.127	0.174	0.211	0.260	0.298	0.335	0.373	0.424
10 min	0.182	0.249	0.302	0.373	0.427	0.480	0.534	0.607
15 min	0.220	0.301	0.365	0.451	0.516	0.581	0.646	0.734
30 min	0.306	0.418	0.508	0.627	0.717	0.808	0.899	1.021
60 min	0.415	0.567	0.689	0.850	0.972	1.094	1.218	1.384
3 hour	0.664	0.874	1.049	1.292	1.484	1.683	1.892	2.184
6 hour	0.889	1.163	1.393	1.714	1.968	2.234	2.515	2.908
12 hour	1.188	1.585	1.910	2.357	2.704	3.059	3.428	3.931
24 hour	1.619	2.190	2.648	3.260	3.722	4.184	4.654	5.280
48 hour	2.005	2.665	3.186	3.870	4.378	4.880	5.381	6.038
72 hour	2.286	3.004	3.566	4.298	4.839	5.369	5.896	6.581
96 hour	2.468	3.217	3.802	4.561	5.118	5.663	6.203	6.902

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.721	0.722	0.723	0.724	0.724	0.722	0.722	0.721
10 min	0.720	0.724	0.723	0.723	0.724	0.723	0.722	0.721
15 min	0.720	0.723	0.725	0.724	0.724	0.723	0.722	0.721
30 min	0.721	0.723	0.724	0.724	0.724	0.723	0.722	0.721
60 min	0.721	0.724	0.724	0.724	0.724	0.723	0.723	0.721
3 hour	0.719	0.721	0.723	0.723	0.724	0.724	0.724	0.723
6 hour	0.718	0.720	0.722	0.723	0.723	0.723	0.724	0.724
12 hour	0.717	0.720	0.722	0.723	0.724	0.724	0.724	0.725
24 hour	0.716	0.720	0.722	0.723	0.723	0.724	0.724	0.724
48 hour	0.714	0.717	0.719	0.720	0.721	0.721	0.721	0.722
72 hour	0.713	0.716	0.717	0.718	0.719	0.719	0.720	0.720
96 hour	0.712	0.714	0.715	0.716	0.717	0.717	0.717	0.718

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.092	0.126	0.153	0.188	0.216	0.242	0.269	0.306
10 min	0.131	0.180	0.218	0.270	0.309	0.347	0.386	0.438
15 min	0.158	0.218	0.265	0.327	0.374	0.420	0.466	0.529
30 min	0.221	0.302	0.368	0.454	0.519	0.584	0.649	0.736
60 min	0.299	0.411	0.499	0.615	0.704	0.791	0.881	0.998
3 hour	0.477	0.630	0.758	0.934	1.074	1.218	1.370	1.579
6 hour	0.638	0.837	1.006	1.239	1.423	1.615	1.821	2.105
12 hour	0.852	1.141	1.379	1.704	1.958	2.215	2.482	2.850
24 hour	1.159	1.577	1.912	2.357	2.691	3.029	3.369	3.823
48 hour	1.432	1.911	2.291	2.786	3.157	3.518	3.880	4.359
72 hour	1.630	2.151	2.557	3.086	3.479	3.860	4.245	4.738
96 hour	1.757	2.297	2.718	3.266	3.670	4.060	4.448	4.956

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: MDZ

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.148	0.191	0.227	0.277	0.318	0.360	0.405	0.468
10 min	0.212	0.274	0.325	0.398	0.455	0.516	0.580	0.671
15 min	0.257	0.331	0.393	0.481	0.551	0.624	0.702	0.812
30 min	0.359	0.463	0.550	0.672	0.769	0.872	0.981	1.134
60 min	0.479	0.618	0.734	0.897	1.027	1.164	1.309	1.514
3 hour	0.783	0.978	1.144	1.383	1.577	1.784	2.007	2.327
6 hour	1.073	1.328	1.544	1.851	2.099	2.360	2.640	3.037
12 hour	1.444	1.792	2.081	2.481	2.795	3.119	3.458	3.927
24 hour	1.980	2.472	2.870	3.406	3.815	4.229	4.652	5.222
48 hour	2.472	3.087	3.574	4.216	4.695	5.171	5.648	6.277
72 hour	2.848	3.558	4.114	4.837	5.371	5.895	6.416	7.095
96 hour	3.154	3.941	4.553	5.343	5.920	6.483	7.039	7.756

Urban Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.623	0.620	0.620	0.619	0.618	0.618	0.617	0.619
10 min	0.622	0.618	0.620	0.618	0.618	0.618	0.618	0.619
15 min	0.623	0.619	0.617	0.618	0.617	0.617	0.617	0.619
30 min	0.624	0.619	0.619	0.618	0.618	0.618	0.619	0.619
60 min	0.623	0.619	0.617	0.617	0.617	0.616	0.617	0.618
3 hour	0.623	0.621	0.619	0.618	0.618	0.618	0.618	0.618
6 hour	0.625	0.622	0.621	0.619	0.619	0.618	0.618	0.617
12 hour	0.625	0.622	0.620	0.617	0.616	0.615	0.614	0.614
24 hour	0.625	0.621	0.618	0.616	0.615	0.613	0.612	0.611
48 hour	0.624	0.621	0.619	0.618	0.617	0.615	0.615	0.614
72 hour	0.625	0.623	0.621	0.620	0.618	0.618	0.617	0.616
96 hour	0.626	0.624	0.623	0.622	0.621	0.620	0.619	0.618

Urban Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.092	0.118	0.141	0.171	0.197	0.222	0.250	0.290
10 min	0.132	0.169	0.202	0.246	0.281	0.319	0.358	0.415
15 min	0.160	0.205	0.242	0.297	0.340	0.385	0.433	0.503
30 min	0.224	0.287	0.340	0.415	0.475	0.539	0.607	0.702
60 min	0.298	0.383	0.453	0.553	0.634	0.717	0.808	0.936
3 hour	0.488	0.607	0.708	0.855	0.975	1.103	1.240	1.438
6 hour	0.671	0.826	0.959	1.146	1.299	1.458	1.632	1.874
12 hour	0.903	1.115	1.290	1.531	1.722	1.918	2.123	2.411
24 hour	1.238	1.535	1.774	2.098	2.346	2.592	2.847	3.191
48 hour	1.543	1.917	2.212	2.605	2.897	3.180	3.474	3.854
72 hour	1.780	2.217	2.555	2.999	3.319	3.643	3.959	4.371
96 hour	1.974	2.459	2.837	3.323	3.676	4.019	4.357	4.793

FRENCH CAMP SLOUGH

NOAA14 Precipitation Frequency Depths

Rainfall Zone: MDZ

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.148	0.191	0.227	0.277	0.318	0.360	0.405	0.468
10 min	0.212	0.274	0.325	0.398	0.455	0.516	0.580	0.671
15 min	0.257	0.331	0.393	0.481	0.551	0.624	0.702	0.812
30 min	0.359	0.463	0.550	0.672	0.769	0.872	0.981	1.134
60 min	0.479	0.618	0.734	0.897	1.027	1.164	1.309	1.514
3 hour	0.783	0.978	1.144	1.383	1.577	1.784	2.007	2.327
6 hour	1.073	1.328	1.544	1.851	2.099	2.360	2.640	3.037
12 hour	1.444	1.792	2.081	2.481	2.795	3.119	3.458	3.927
24 hour	1.980	2.472	2.870	3.406	3.815	4.229	4.652	5.222
48 hour	2.472	3.087	3.574	4.216	4.695	5.171	5.648	6.277
72 hour	2.848	3.558	4.114	4.837	5.371	5.895	6.416	7.095
96 hour	3.154	3.941	4.553	5.343	5.920	6.483	7.039	7.756

Farmington Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.655	0.653	0.652	0.651	0.649	0.650	0.649	0.651
10 min	0.655	0.651	0.652	0.651	0.650	0.650	0.650	0.650
15 min	0.657	0.652	0.649	0.650	0.649	0.649	0.649	0.650
30 min	0.656	0.652	0.651	0.649	0.650	0.650	0.650	0.651
60 min	0.657	0.652	0.649	0.649	0.649	0.649	0.649	0.649
3 hour	0.656	0.653	0.651	0.649	0.649	0.649	0.649	0.649
6 hour	0.656	0.653	0.651	0.650	0.649	0.648	0.648	0.647
12 hour	0.656	0.652	0.649	0.647	0.646	0.644	0.643	0.642
24 hour	0.656	0.651	0.649	0.646	0.644	0.643	0.642	0.640
48 hour	0.655	0.652	0.649	0.648	0.646	0.645	0.644	0.643
72 hour	0.655	0.653	0.651	0.649	0.648	0.647	0.646	0.645
96 hour	0.656	0.654	0.653	0.651	0.650	0.650	0.649	0.648

Farmington Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.097	0.125	0.148	0.180	0.206	0.234	0.263	0.305
10 min	0.139	0.178	0.212	0.259	0.296	0.335	0.377	0.436
15 min	0.169	0.216	0.255	0.313	0.358	0.405	0.456	0.528
30 min	0.236	0.302	0.358	0.436	0.500	0.567	0.638	0.738
60 min	0.315	0.403	0.476	0.582	0.667	0.755	0.850	0.983
3 hour	0.514	0.639	0.745	0.898	1.023	1.158	1.303	1.510
6 hour	0.704	0.867	1.005	1.203	1.362	1.529	1.711	1.965
12 hour	0.947	1.168	1.351	1.605	1.806	2.009	2.223	2.521
24 hour	1.299	1.609	1.863	2.200	2.457	2.719	2.987	3.342
48 hour	1.619	2.013	2.320	2.732	3.033	3.335	3.637	4.036
72 hour	1.865	2.323	2.678	3.139	3.480	3.814	4.145	4.576
96 hour	2.069	2.577	2.973	3.478	3.848	4.214	4.568	5.026

FRENCH CAMP SLOUGH

NOAA14 Precipitation Frequency Depths

Rainfall Zone: MDZ

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.148	0.191	0.227	0.277	0.318	0.360	0.405	0.468
10 min	0.212	0.274	0.325	0.398	0.455	0.516	0.580	0.671
15 min	0.257	0.331	0.393	0.481	0.551	0.624	0.702	0.812
30 min	0.359	0.463	0.550	0.672	0.769	0.872	0.981	1.134
60 min	0.479	0.618	0.734	0.897	1.027	1.164	1.309	1.514
3 hour	0.783	0.978	1.144	1.383	1.577	1.784	2.007	2.327
6 hour	1.073	1.328	1.544	1.851	2.099	2.360	2.640	3.037
12 hour	1.444	1.792	2.081	2.481	2.795	3.119	3.458	3.927
24 hour	1.980	2.472	2.870	3.406	3.815	4.229	4.652	5.222
48 hour	2.472	3.087	3.574	4.216	4.695	5.171	5.648	6.277
72 hour	2.848	3.558	4.114	4.837	5.371	5.895	6.416	7.095
96 hour	3.154	3.941	4.553	5.343	5.920	6.483	7.039	7.756

Upper Watershed Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.718	0.717	0.717	0.716	0.715	0.714	0.713	0.714
10 min	0.717	0.716	0.717	0.716	0.716	0.714	0.714	0.713
15 min	0.719	0.716	0.716	0.715	0.714	0.714	0.713	0.713
30 min	0.719	0.717	0.716	0.715	0.715	0.715	0.714	0.713
60 min	0.719	0.717	0.715	0.715	0.714	0.714	0.714	0.713
3 hour	0.717	0.716	0.715	0.715	0.715	0.714	0.714	0.714
6 hour	0.716	0.715	0.715	0.714	0.714	0.713	0.713	0.713
12 hour	0.715	0.714	0.713	0.712	0.712	0.711	0.711	0.710
24 hour	0.714	0.713	0.712	0.711	0.710	0.709	0.709	0.708
48 hour	0.712	0.711	0.711	0.710	0.710	0.709	0.708	0.708
72 hour	0.711	0.711	0.710	0.710	0.709	0.709	0.708	0.708
96 hour	0.711	0.711	0.710	0.710	0.710	0.709	0.709	0.709

Upper Watershed Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.106	0.137	0.163	0.198	0.227	0.257	0.289	0.334
10 min	0.152	0.196	0.233	0.285	0.326	0.368	0.414	0.478
15 min	0.185	0.237	0.281	0.344	0.393	0.446	0.501	0.579
30 min	0.258	0.332	0.394	0.480	0.550	0.623	0.700	0.809
60 min	0.344	0.443	0.525	0.641	0.733	0.831	0.935	1.079
3 hour	0.561	0.700	0.818	0.989	1.128	1.274	1.433	1.661
6 hour	0.768	0.950	1.104	1.322	1.499	1.683	1.882	2.165
12 hour	1.032	1.279	1.484	1.766	1.990	2.218	2.459	2.788
24 hour	1.414	1.763	2.043	2.422	2.709	2.998	3.298	3.697
48 hour	1.760	2.195	2.541	2.993	3.333	3.666	3.999	4.444
72 hour	2.025	2.530	2.921	3.434	3.808	4.180	4.543	5.023
96 hour	2.242	2.802	3.233	3.794	4.203	4.596	4.991	5.499

FRENCH CAMP SLOUGH
NOAA14 Precipitation Frequency Depths
Rainfall Zone: MDZ

Calculated Average Depths for Rainfall Zone [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.148	0.191	0.227	0.277	0.318	0.360	0.405	0.468
10 min	0.212	0.274	0.325	0.398	0.455	0.516	0.580	0.671
15 min	0.257	0.331	0.393	0.481	0.551	0.624	0.702	0.812
30 min	0.359	0.463	0.550	0.672	0.769	0.872	0.981	1.134
60 min	0.479	0.618	0.734	0.897	1.027	1.164	1.309	1.514
3 hour	0.783	0.978	1.144	1.383	1.577	1.784	2.007	2.327
6 hour	1.073	1.328	1.544	1.851	2.099	2.360	2.640	3.037
12 hour	1.444	1.792	2.081	2.481	2.795	3.119	3.458	3.927
24 hour	1.980	2.472	2.870	3.406	3.815	4.229	4.652	5.222
48 hour	2.472	3.087	3.574	4.216	4.695	5.171	5.648	6.277
72 hour	2.848	3.558	4.114	4.837	5.371	5.895	6.416	7.095
96 hour	3.154	3.941	4.553	5.343	5.920	6.483	7.039	7.756

Average Storm Centering
Area Reduction Factors

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.687	0.685	0.685	0.684	0.682	0.682	0.681	0.683
10 min	0.686	0.684	0.685	0.684	0.683	0.682	0.682	0.682
15 min	0.688	0.684	0.683	0.683	0.682	0.682	0.681	0.682
30 min	0.688	0.685	0.684	0.682	0.683	0.683	0.682	0.682
60 min	0.688	0.685	0.682	0.682	0.682	0.682	0.682	0.681
3 hour	0.687	0.685	0.683	0.682	0.682	0.682	0.682	0.682
6 hour	0.686	0.684	0.683	0.682	0.682	0.681	0.681	0.680
12 hour	0.686	0.683	0.681	0.680	0.679	0.678	0.677	0.676
24 hour	0.685	0.682	0.681	0.679	0.677	0.676	0.676	0.674
48 hour	0.684	0.682	0.680	0.679	0.678	0.677	0.676	0.676
72 hour	0.683	0.682	0.681	0.680	0.679	0.678	0.677	0.677
96 hour	0.684	0.683	0.682	0.681	0.680	0.680	0.679	0.679

Average Storm Centering
Area Reduced Rainfall Depths [inches]

Duration	Frequency [years]							
	2	5	10	25	50	100	200	500
5 min	0.102	0.131	0.155	0.189	0.217	0.246	0.276	0.320
10 min	0.145	0.187	0.223	0.272	0.311	0.352	0.396	0.458
15 min	0.177	0.226	0.268	0.329	0.376	0.426	0.478	0.554
30 min	0.247	0.317	0.376	0.458	0.525	0.596	0.669	0.773
60 min	0.330	0.423	0.501	0.612	0.700	0.794	0.893	1.031
3 hour	0.538	0.670	0.781	0.943	1.076	1.217	1.369	1.587
6 hour	0.736	0.908	1.055	1.262	1.432	1.607	1.798	2.065
12 hour	0.991	1.224	1.417	1.687	1.898	2.115	2.341	2.655
24 hour	1.356	1.686	1.954	2.313	2.583	2.859	3.145	3.520
48 hour	1.691	2.105	2.430	2.863	3.183	3.501	3.818	4.243
72 hour	1.945	2.427	2.802	3.289	3.647	3.997	4.344	4.803
96 hour	2.157	2.692	3.105	3.639	4.026	4.408	4.779	5.266

Attachment 6- H. ITR Comment Forms for French Camp Slough HEC-HMS Modeling

Lower San Joaquin River Feasibility Study

HYDROLOGIC MODELING REVIEW –FRENCH CAMP SLOUGH WATERSHED

Task Order 6 – LSJRFS Work-In-Kind Hydrology

Reviewer: Domenichelli & Associates
Review Date: 11-23-10
PBI Response Date: 12-21-10
DA Backcheck: 01-04-11

Memorandum Comments:

1. Section 6.4.3 Reservoirs: In the 5th paragraph the memorandum states that the rating curves were estimated by entering geometries into HEC-RAS. Consider providing the results as an appendix

   PBI Response: Hydraulic calculations associated with assigning reservoir storage/discharge relationships are now included in Attachment B.

   DA Response: Accepted

2. Section 6.4.4 Diversions: The memorandum states that 'In all cases, diversion flows were proportionally based on channel geometries'. This is true in all cases except for Duck Creek. The Duck Creek bifurcation has a structure to control the flow diverted to Littlejohns Creek.

   PBI Response: Section 6.4.4 has been updated.

   DA Response: Accepted

3. Section 6.5 Model Calibration:
   a. An adjustment factor of 0.60 for the constant loss rates is large. When comparing sub-basins comprised entirely of soil group D for the French Camp Slough (0.015 after adjustment) to Calaveras River (0.023 after adjustment) soil loss rates are decreased by 35%. Given the proximity of Calaveras River to Duck Creek this difference is significant. When considering the effect of a 0.6 adjustment factor it would convert the PBI assumed loss rate for a type B soil from the typical range (per table 2) to the range for a Type C soil. This is not consistent with the use of NRCS soils data for the basis of assigning loss rates.

PBI Response: See response to 3(b). An adjustment factor of 0.6 is no longer used.

DA Response: Accepted

b. Use of the Lone Tree Creek gage to determine the entire watershed's soil loss rates is the cause for the large adjustment factor. The Lone Tree Creek gage considers only a handful of sub-basins of similar condition and may not accurately reflect the losses in the upper watershed of Littlejohns creek. Additionally, the method of using a HEC-RAS model extended to ALERT Gage 205 (Lone Tree Creek gage) produces an unknown amount of possible error when converting river stage data to flow. Consider using an adjustment factor and corresponding loss rates similar to the Calaveras River modeling effort where more reliable stream flow data was used.

PBI Response: Agreed. There are too many uncertainties associated with the current calibration. There was very little concurrent rainfall/runoff data to choose from in the French Camp watershed and no rating curve had been established for ALERT Gage 205.

The French Camp Slough's subbasins now use an adjustment factor of 0.85 which was established through calibration of the neighboring Calaveras River watershed.

Section 6.5 has been updated.

DA Response: Accepted

c. Model Sub-basins where farming is prevalent can be expected to have higher loss rates and additional ponding than sub-basins in the eastern portion of the watershed. There are many flat areas where water ponds up to a foot of depth in a field before it discharges into drainage ditches. Consider higher loss rates for the farming areas. This is discussed further in the Model Comments section.

PBI Response: See response to Model Comment #4.

DA Response: Accepted

d. Figure 8: The "projected data points" are unnecessary for the calibration of the model. Consider removing the points from the graph.

PBI Response: See response to 3(b). The calibration technique has been modified and this figure has now been removed.

DA Response: Accepted

Model Comments:

4. In the original Tidewater HEC-HMS model sub-basin sizes were reduced based on the estimated percentage of the sub-basin that would not drain. This reduction in the sub-basin size was done to model the effects of ponding in the fields (especially on farms required to retain all runoff. ie. dairies). The new model re-established basin sizes but did not take into consideration the portions of the basin which do not drain. The element description states the percentage of the basin which does not drain in both the Tidewater model and the new FCS model.

   PBI Response: PBI's calculated subbasin areas now take in to consideration the percent of area estimated to be isolated. These percentages are based on field investigations conducted for the Tidewater Model and are now presented in Appendix A.

   DA Response: The table in Appendix A does not match the percentage of 'No Drain' listed in the subbasin description in the FCS model or the Tidewater model (Ex: basin LT B4 in the model lists 10% no drain but in Appendix A there is no LT B4). Check Appendix A for consistency with the model element description.

   PBI Response: The 'No Drain' subbasin descriptions were removed from the PBI Model. These descriptions were left over from the Tidewater Model and were not up to date.

5. The French Camp Slough model has many storage areas and diversions where the channels encounter embankments due to highways and railroads. Any increase in model flow due to lower loss rates gets stored in these storage areas. The storage upstream of the highways and railroads is increased. The new model has nearly doubled the storage calculated in the Tidewater model which would result in more extensive flooding in areas such as at Highway 99. Highway 99 may no longer be a zone x and the FEMA maps will be expanded in all areas where ponding occurs. As stated in previous comments relative to loss rates, the 0.6 adjustment factor results in significant impacts to the flood plain and should be analyzed in more detail.

   PBI Response: See response to 3(b). An adjustment factor of 0.6 is no longer used.

   DA Response: Accepted

6. Consider changing the datum to NAVD88 for the HEC-HMS storage elevation curves and the future HEC-RAS model. FEMA maps updated in 2009 for the FCS project area are in NAVD88. The previous hydrologic model and FEMA maps were in NGVD29.

PBI Response: Agreed. All elevation-storage functions were converted from NGVD29 to NAVD88 using CORPSCON software. The conversion is now mentioned in *Section 6.4.3: Reservoirs* and a CORPSCON output table is included in Attachment F.

DA Response: Accepted

Attachment 6- I. SPK Comment Forms for French Camp Slough HEC-HMS Modeling

Corps of Engineers, Hydrology Section, Review of French Camp Slough HEC-1 to HEC-HMS model conversion and preliminary report.

31 January 2011

Steven F. Holmstrom, P.E.

The Technical Memorandum for the Lower San Joaquin River Feasibility Study French Camp Slough HEC-HMS modeling DRAFT hydrology report has been reviewed and the following comments are provided.

22. It should be noted in paragraph 6.1.3 Topography that the vertical elevation datum used is NAVD (1988).

    PBI Response: Agreed.

23. The Design Storms procedure described in paragraph 6.3 should reflect the guidance in the "Storm Distribution Procedure" transmitted on January 21, 2011. It is noted that this guidance has not been reviewed and accepted by Peterson-Brustad, Inc. However, upon review and acceptance by all members of the study team, that procedure should be integrated into the report(s).

    PBI Response: Agreed.

24. In paragraph 6.4.3 Reservoirs, the firm name in the fourth paragraph in that section should be changed to "David Ford Consulting Engineers, Inc.".

    PBI Response: Agreed.

Attachment 7- A. PBI Internal Review Comments and Responses

PBI Internal Review Comment / Response Log

PROJECT: **Lower San Joaquin River Feasibility Study**
REVIEW DOCUMENT(S): **Draft F3 Hydrology Appendix**

DATE: **7/11/2012**

SUBMITTED BY: **Michael Rossiter, PE, CFM**
REVIEWER: **Dave Peterson, PE**

REFERENCE			COMMENT				RESPONSE				
			Comment Codes: **M**=Mandatory Response; **S**=Suggested Correction; **Q**=Question; **G**=General Comment;				Response Codes: **A**=Agree, will revise; **D**=Disagree, see explanation; **F**=Follow up required; **G**=General Response				
Comment No.	Dwg/Sec	Page/Sht	Code	Description	By	Date	Code	Explanation	By	Date	Backcheck By/Date
1	2.0	11	**M**	Explain why the 72hr design storm was used.	DAP	7/7/12	**A**	Section 2.0 introduction is updated	MJR	7/11/12	DAP / 7/11/12
2	2.3	13	**M**	Explain what you mean by a "standard" 24 hr storm that you compared results with	DAP	7/7/12	**A**	Section 2.3 is updated	MJR	7/11/12	DAP / 7/11/12
3	3.3.1	19	**M**	Need to discuss how basins outside of levees without pump stations are treated. How are they modeled?	DAP	7/7/12	**A**	Section 3.3.1 updated.	MJR	7/11/12	DAP / 7/11/12
4	3.7	36	**M**	Table 3-5 is confusing. Reorganize table to show that you ran all AEP events for all development and storm centering conditions.	DAP	7/7/12	**A**	Table updated.	MJR	7/11/12	DAP / 7/11/12
5	4.3.2	44	**M**	Need more detail on flow split in Paragraph 1.	DAP	7/7/12	**A**	Section 4.3.2 updated.	MJR	7/11/12	DAP / 7/11/12
6	4.3.2	44	**M**	Add pump station location on to Figure 4-2.	DAP	7/7/12	**A**	Figure updated.	MJR	7/11/12	DAP / 7/11/12
7	4.7	58	**M**	Table 4-5 is confusing. Reorganize table to show that you ran all AEP events for all development and storm centering conditions.	DAP	7/7/12	**A**	Table updated.	MJR	7/11/12	DAP / 7/11/12
8	4.7.2	58	**M**	Explain further why you assigned this equivalent record length for the Mosher model	DAP	7/7/12	**A**	Section 4.7.2	MJR	7/11/12	DAP / 7/11/12
9	4.7.1	58	**M**	Tables 4-6 and 4-7 need to include results at the mouth (after Atlas Tract)	DAP	7/7/12	**A**	Tables updated.	MJR	7/11/12	DAP / 7/11/12
10	5.3.1	65	**M**	Need to discuss how basins outside of levees without pump stations are treated. How are they modeled?	DAP	7/7/12	**A**	Section 5.3.1 updated.	MJR	7/11/12	DAP / 7/11/12
11	5.3.1	66	**M**	Add location of Bellota Dam to Figure 5-2	DAP	7/7/12	**A**	Figure updated.	MJR	7/11/12	DAP / 7/11/12
12	5.7	84	**M**	Table 5-7 is confusing. Reorganize table to show that you ran all AEP events for all development and storm centering conditions.	DAP	7/7/12	**A**	Table updated.	MJR	7/11/12	DAP / 7/11/12
13	5.7.1	86	**M**	To avoid confusion, only report results for the Average storm centering	DAP	7/7/12	**A**	Table updated.	MJR	7/11/12	DAP / 7/11/12
14	6.7	109	**M**	Table 6-6 is confusing. Reorganize table to show that you ran all AEP events for all development and storm centering conditions.	DAP	7/7/12	**A**	Table updated.	MJR	7/11/12	DAP / 7/11/12
15	6.7.1	111	**M**	To avoid confusion, only report results for the Average storm centering	DAP	7/7/12	**A**	Tables updated.	MJR	7/11/12	DAP / 7/11/12

Attachment 8- A. SPK Review of Draft F3 Hydrology Appendix

Corps of Engineers, Hydrology Section, Review of LSJRFS Draft F3 Hydrology Report dated 07122012
16 July 2012
Steven F. Holmstrom, P.E.

PBI Responses: 26 July 2012, Michael Rossiter, P.E.

The draft F3 Hydrology Report for the Lower San Joaquin River feasibility study has been reviewed and the following comments are provided.

1. Sections 3.1.2, 4.1.2, 5.1.2 add description of vertical datum that was used. Since 30-meter DEM's and DWR LiDAR data are being used there is at least some elevation data used in the watershed.

   PBI Response: Description of vertical datum was added to the listed sections.

2. Section 4.3.2, "… an additional three pumps at 25.1 cfs …". From the table 4-1 it is apparent that the 25.1 cfs is for each pump for a total of 75.3 cfs. The reference in paragraph 4.3.2 must be made clear that the 25.1 cfs is for each pump.

   PBI Response: Section 4.3.2 updated with re-wording of pump description.

3. Table 4-7, the 1/200 AEP flow value is less in the future condition than in the existing condition. That appears to be inconsistent with other values in the table. Verify and correct table 4-7.

   PBI Response: This inconsistency was due to the Atlas Tract pump station which is coded into the future conditions model. The pump station has 4 pumps which were set to sequentially shut off when the inflow to the pump decreases. For the 200-yr event, the timing was such that the inflow to the Atlas Tract pump station decreased right as the main flood wave in Mosher Slough was passing through. So the pump shut off for ~15 minutes during the passing of the main channel peak flow.

   This was corrected by assigning a 60-minute minimum rest and minimum run time for Atlas Tract pumps #2, #3, and #4. All future conditions production runs were re-run and Table 4-7 was updated. The only flows that saw any change were for the 'Mosher Slough u/s of Bear Creek Confluence" location.

   The only changes to the HEC-HMS model was specifying the 60-minute minimum rest/run time for the Atlas Tract pump station. The previous version of the model has been replaced by this updated model on PBI's FTP site.

4. The flow values for the 1/10 and 1/25 AEP events do not match between tables 5-2 and table 5-7. The values must be verified and corrected as required.

   PBI Response: The minor discrepancies between the two tables for the 1/10 and 1/25 AEP events are the result of the overall peak identified in the USACE table (Table 5-2) having occurred outside of the HEC-HMS simulation window (31DEC1996-19JAN1997).
   …
   For example:
   The USACE hydrograph at Bellota has a peak flow of 9,529 cfs for the 1/10 AEP event which occurs at 21DEC1996-17:00. This is what is recorded in Table 5-2.

   The HEC-HMS model's 72hr design storm occurred between 31DEC1996-0:00 and 04JAN1997-0:00; the model simulation was run from31DEC1996-0:00 to 19JAN1997-0:00. The peak flow at Bellota during the model simulation window was 9,388 cfs and occurred on 02JAN1997-16:00. This is what is recorded in Table 5-7.
   …
   To avoid confusion, the 1/10 and 1/25 AEP peak flows in Table 5-2 were revised to match the modeled peak flows listed in Table 5-7. Table 5-2 is now introduced as: "The following table is based on the information in the USACE amendment and shows the flow-frequency relationship for modeled flows at the Bellota control point."
5. Section 3.3.1 and 5.3.1 "…nearly all cases, these basins drain through the culverts before the water surface elevation in the main channel would cause a closure of the headgate." Explain what the exceptions are and how they will or may be handled in the Hydraulic analysis task of the project.

   PBI Response: These paragraphs were re-worded to explain our assumption:

   "For subbasins that are on the outside of a levee which do not have pump stations, runoff is coded to enter the main channel at road crossings where there are through-levee culverts. The assumption is made that the culvert headgates will remain open and allow outside flow to enter the main channel. This assumption was made to remain conservative and to account for the potential replacement of culverts by pump stations in the future."

6. Since the future condition is the same as the existing condition for the Calaveras River, add a footnote (2) to table 5-7 stating that "there is no change from the existing to the future condition, therefore only one table is shown", and change "Existing Conditions" to read "Existing and Future Conditions (2)".

   PBI Response: Footnote added to Table 5-7. Title of Table 5-7 updated.

7. Table 6-6 and 6-7, The flows for Duck Creek at Highway 99 are lower for the Future condition than for the existing condition for the 1/200 and 1/500 AEP events. Verify the flows and correct the table(s) as required.

   PBI Response: There were 3 pump stations (PS-DC4, PS-DC5 and PS-DC6) added along Duck Creek for the future conditions model. These pump stations regulate flow coming from these future-developed subbasins. Regulated, future flows for Duck Creek at Hwy 99 therefore end up being less than the non-regulated, existing conditions flows for the larger 1/200 and 1/500 AEP events.

8. Section 6.7.2, The equivalent record length for the French Camp system is said to be 10-30 years, whereas the equivalent record length for the other basins is said to be 20-30 years. This appears to be inconsistent. Is there less confidence in the French Camp model or does the record length need to be corrected?

   PBI Response: French Camp Slough model parameters were adjusted based on a calibration of the neighboring Calaveras River watershed. Therefore there's slightly less confidence in this model than for the other watersheds. The French Camp flows were categorized as flows that were "estimated with a rainfall-runoff-routing model with regional model parameters" which should have an equivalent record length of 10-30 years according to EM 1619. Section 6.7.2 was updated to clarify this.

   Additional Note: The Mosher Slough model parameters were adjusted based on a calibration of the neighboring Bear Creek watershed, however, Mosher Slough flows are largely dependent on pumped flows with known pump capacities and therefore still have a high level of confidence.

From: Holmstrom, Steven F SPK
Sent: Friday, August 10, 2012 2:07 PM
To: 'Michael Rossiter'
Cc: David Peterson (dpeterson@pbieng.com); Williams, Michelle R SPK; High, John M SPK
Subject: LSJR FS F3 Hydrology Appendix Reviewed and Back-checked (UNCLASSIFIED)
Signed By: steven.f.holmstrom@us.army.mil

Classification: UNCLASSIFIED
Caveats: NONE

Mike,

I have reviewed the comments and responses for the subject report as documented in attachment 8-A in the report.

I have no additional comments and I believe that all comments and responses are back-checked and resolved.

This is a fine report. Thank you for the effort.

Michelle and PDT: the final report has been copied to the "\\Amethyst\Projects\" drive in the following directory:
-\LSJRFS\H&H\Hydrology\LSJRFS Hydrology Report_v4_073012.pdf.

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Classification: UNCLASSIFIED
Caveats: NONE